INDIAN JOURNAL OF POWER & RIVER VALLEY DEVELOPMENT

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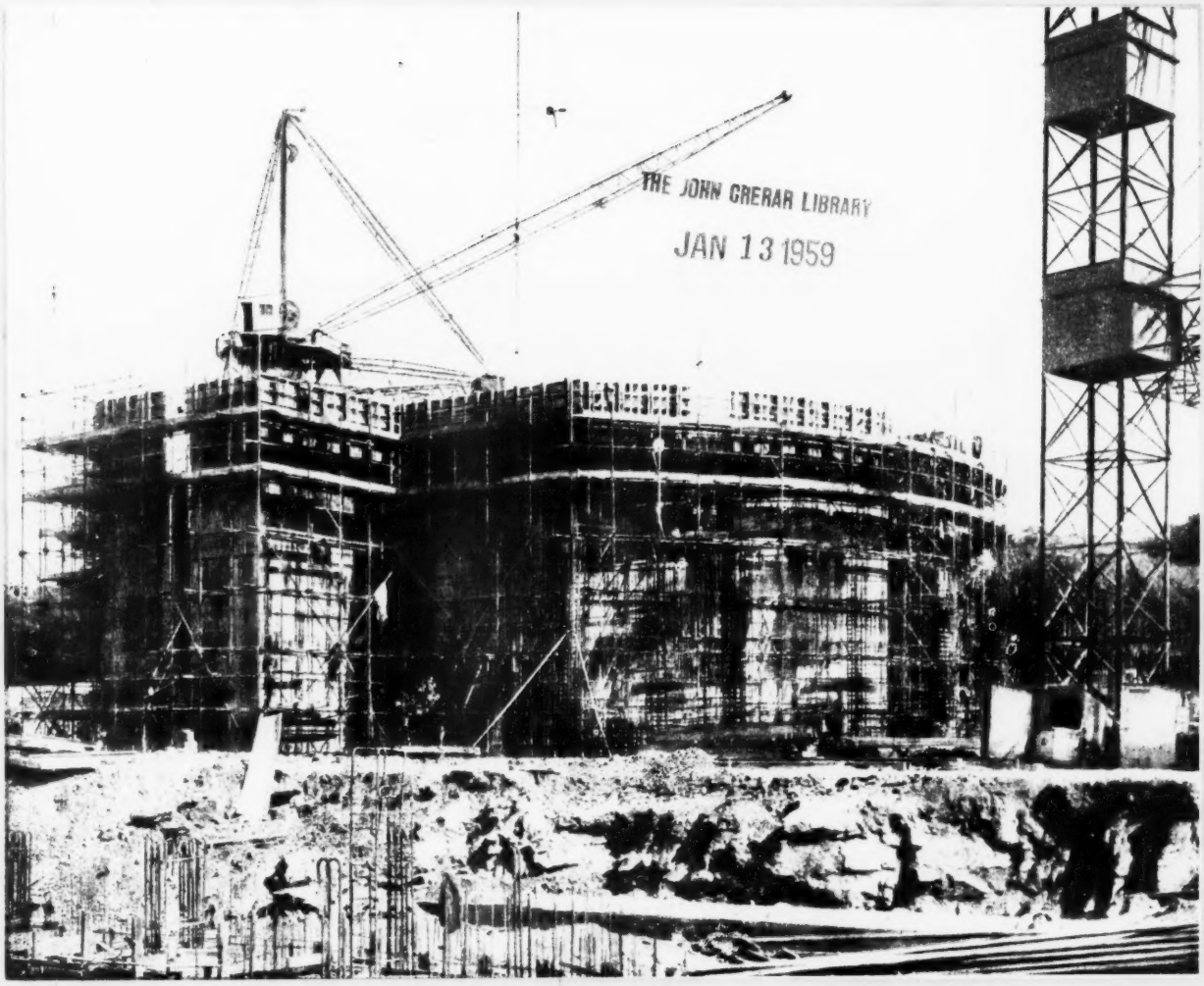
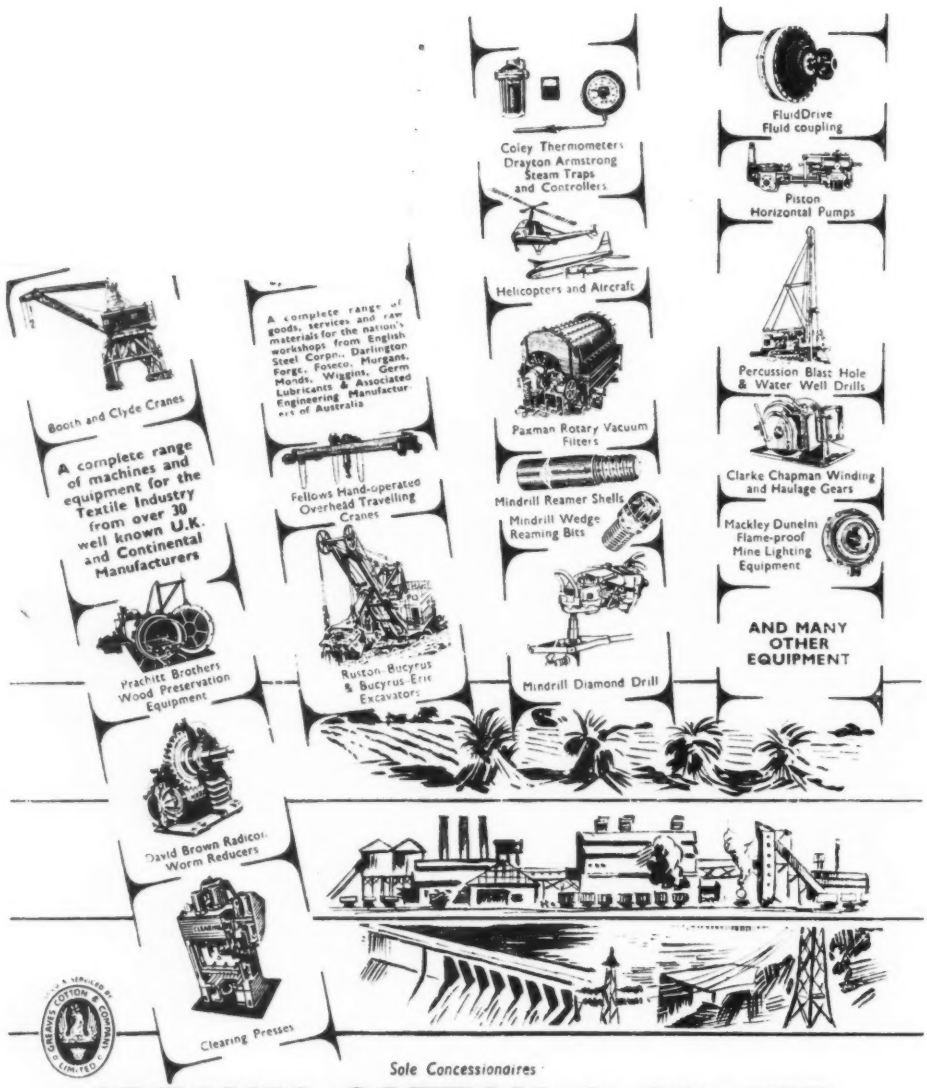
VOL. VIII. NO. 10 OCTOBER, 1958 RUPEES TWO



HUNTERSTON NUCLEAR GENERATING STATION UNDER CONSTRUCTION, U. K.

'ion to help itself...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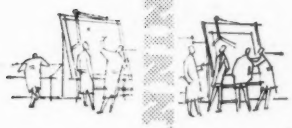

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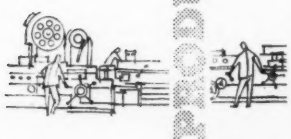

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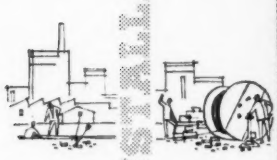

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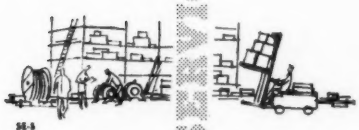

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Another emblem, the one shown above, will soon be known throughout the technical world of India. It will be a hallmark of quality for motors, transformers, switchgear, cables and meters soon to be built under Licence in this country in cooperation with noted Indian industrial enterprises.

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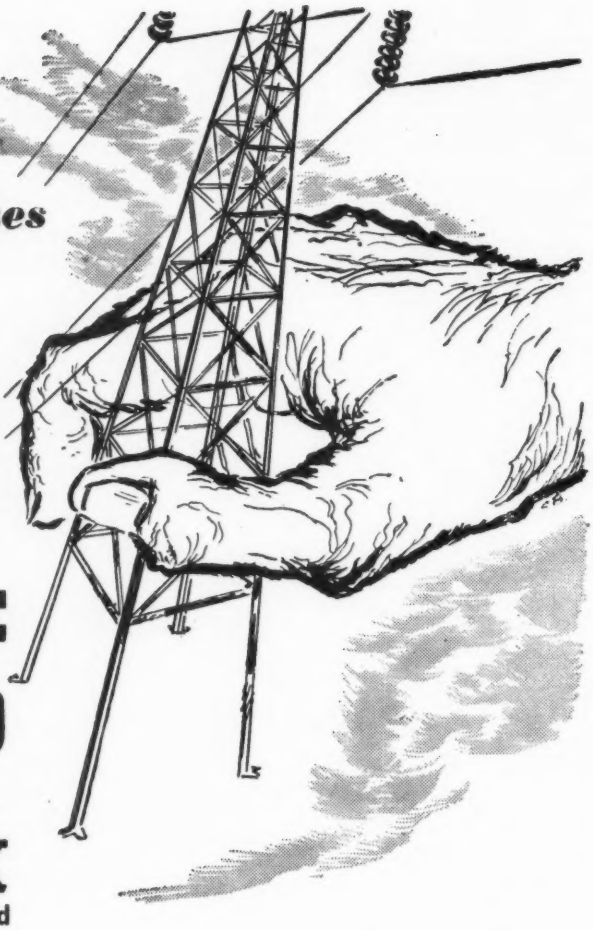

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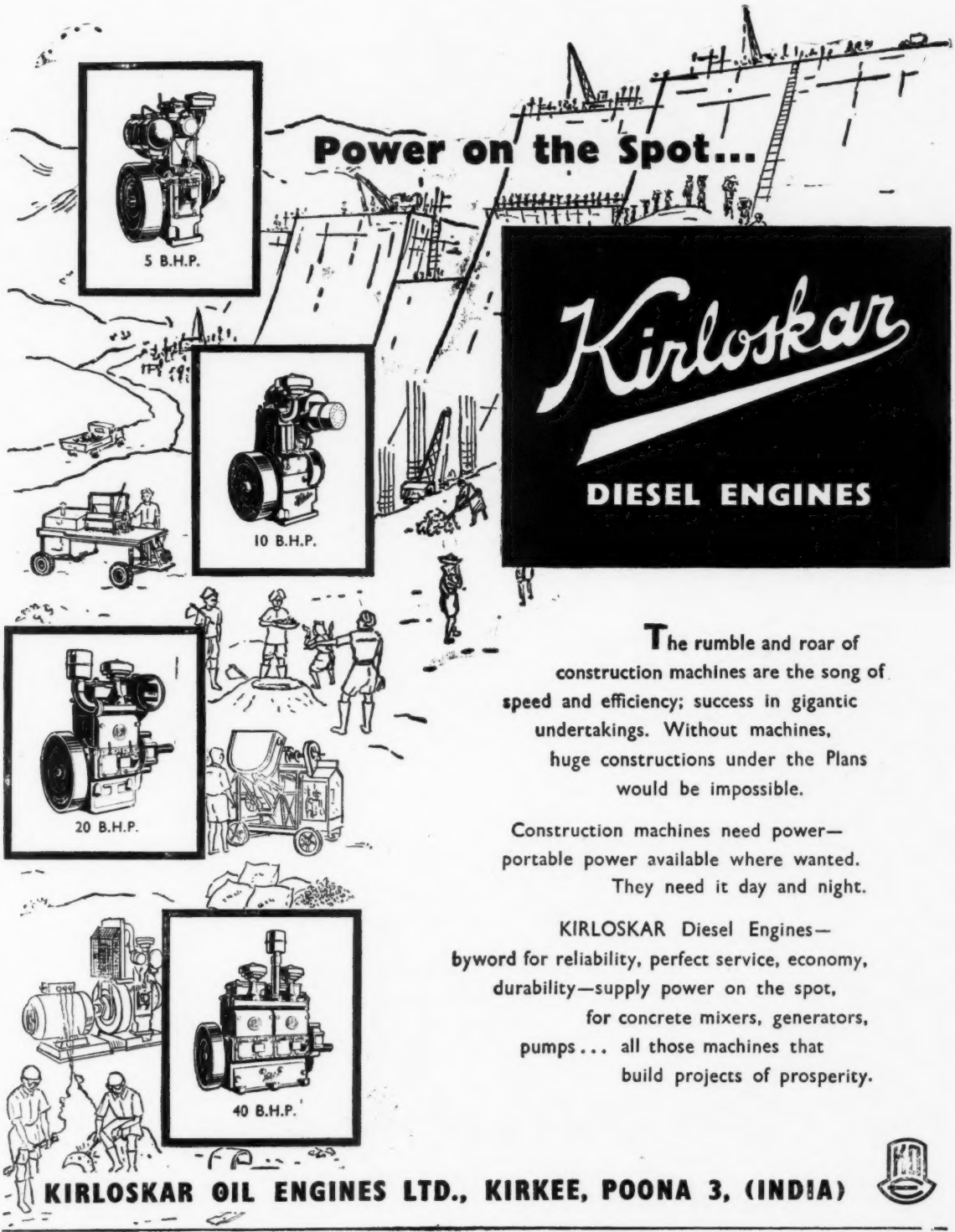

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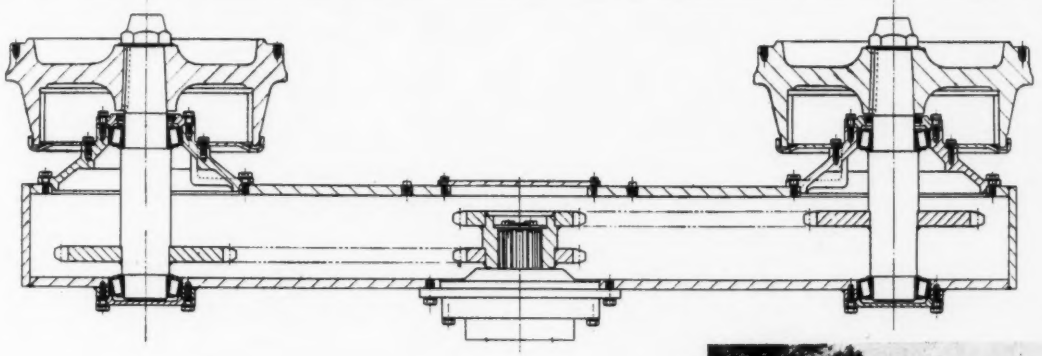

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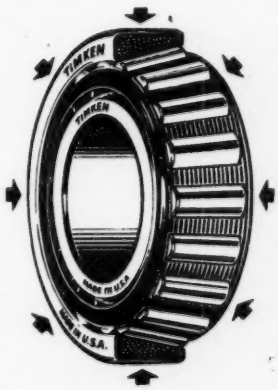

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Aristotle had once described
his pupil, Alexander the Great,
as many in one : man of action,
man of logic, romantic dreamer,
believer in magic and so on.
Like Alexander, we too
have become many in one.
At birth, we were
christened Textile Machinery
Corporation Limited, or
makers of textile machinery,
and over the years we have
made many such machines.
But we did not stop here.
The 4,500 strong Texmaco,
today manufactures a wide range
of engineering products—cotton
and jute textile machinery,
industrial and locomotive
boilers, pressure vessels, steel
structurals, railway rolling
stock, sugar mill machinery
steel castings and so on. With
the implementation of our
new projects, now under
way, we would become yet
many more in one.
Whatever Alexander did,
he was inspired by the
thought of adding fresh
glory to the name of
Macedon. Whatever we
might do, there is a
single aim that directs
our activities—the proper
utilisation of all our
resources, to manufacture
quality machinery—
machinery to help India
industrialise and bring
prosperity to her teeming
millions.

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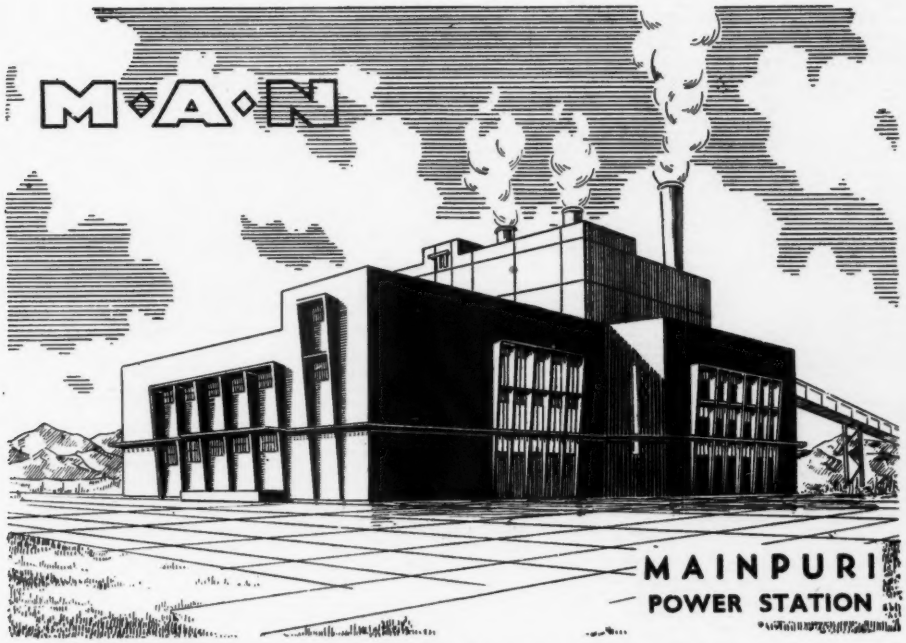

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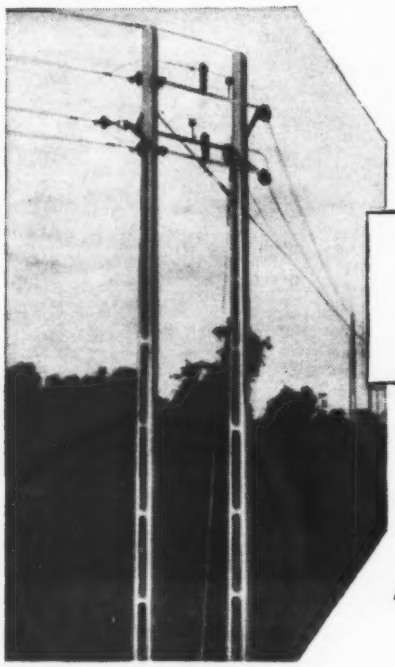

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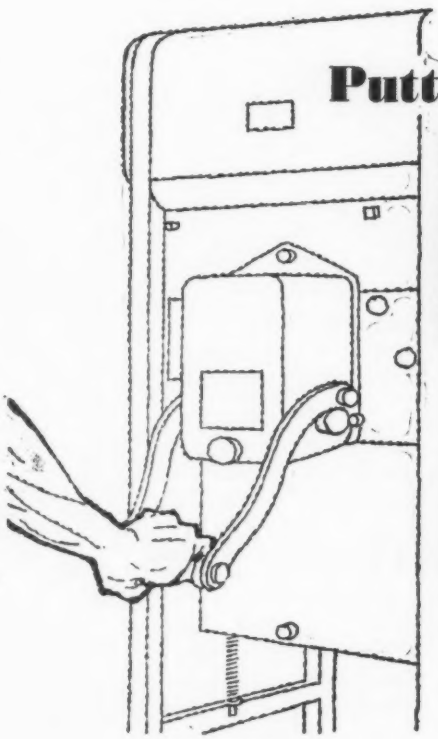
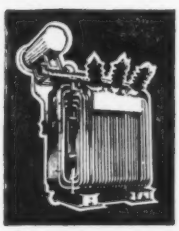
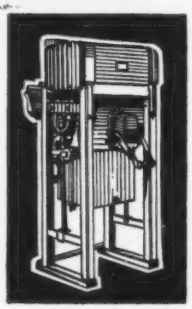
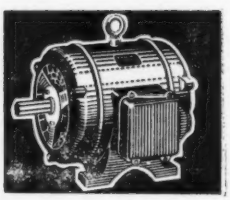


Putting electricity to work







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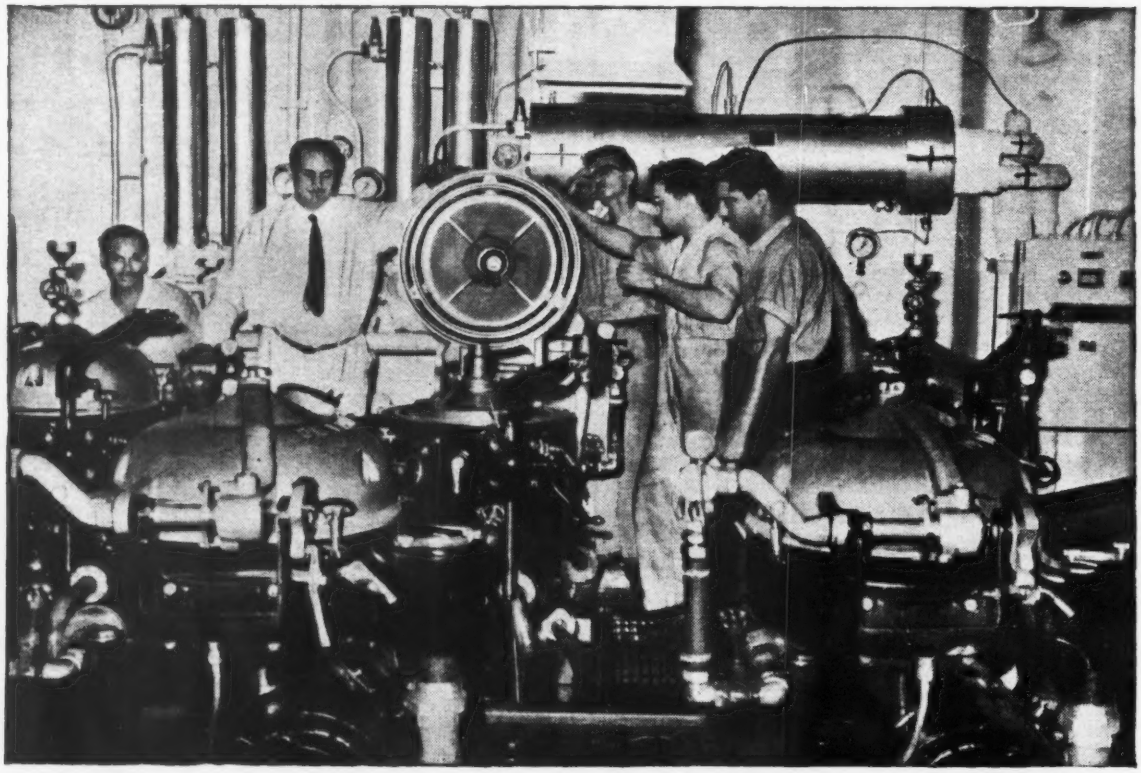

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resist internal and external corrosion

remain in position during manufacture and subsequent storage.

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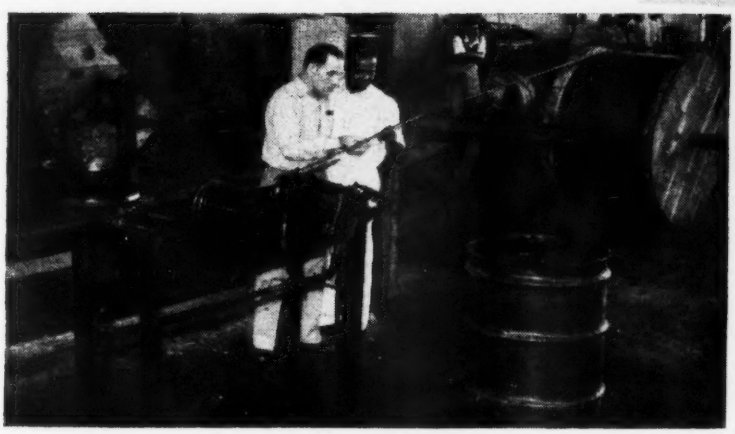

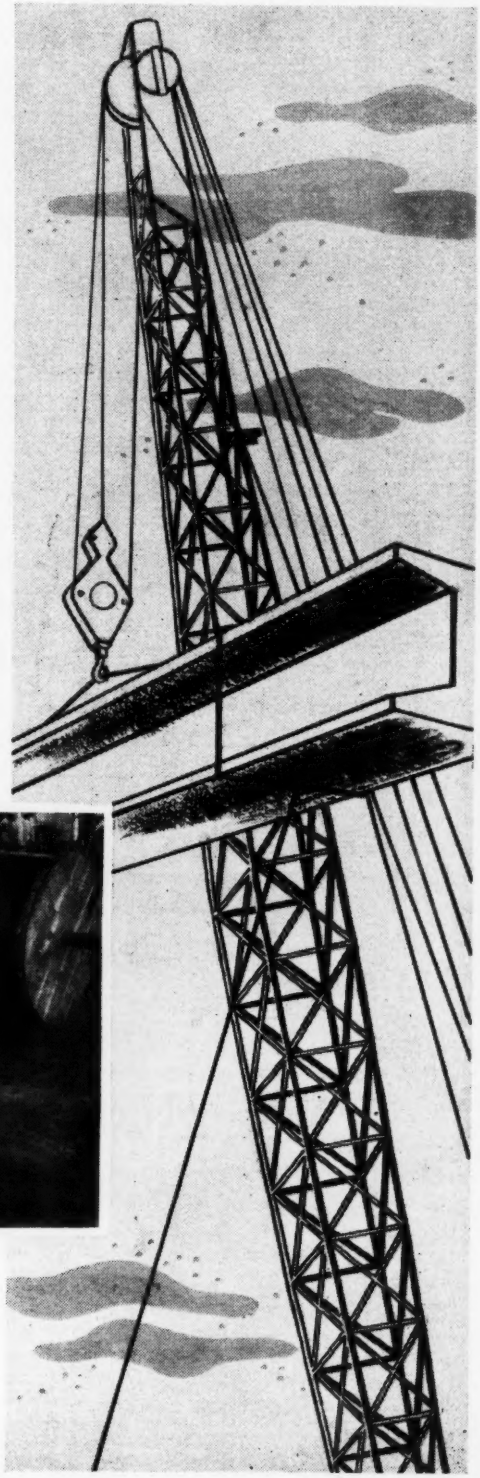

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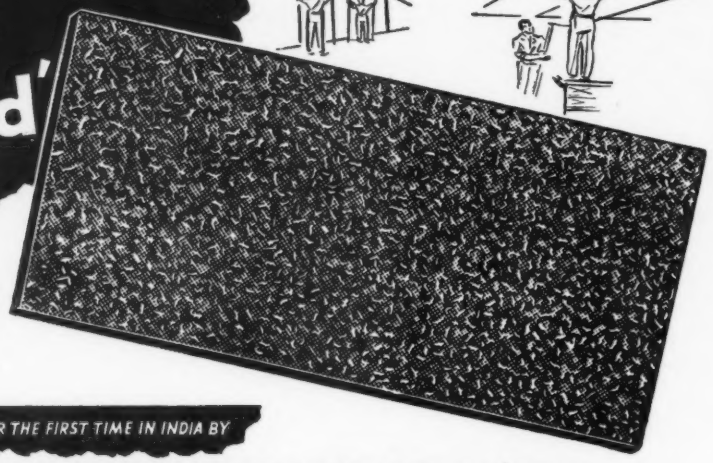

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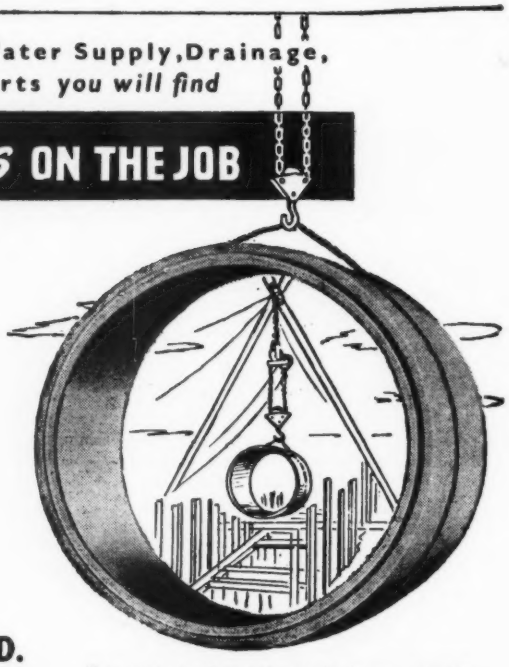

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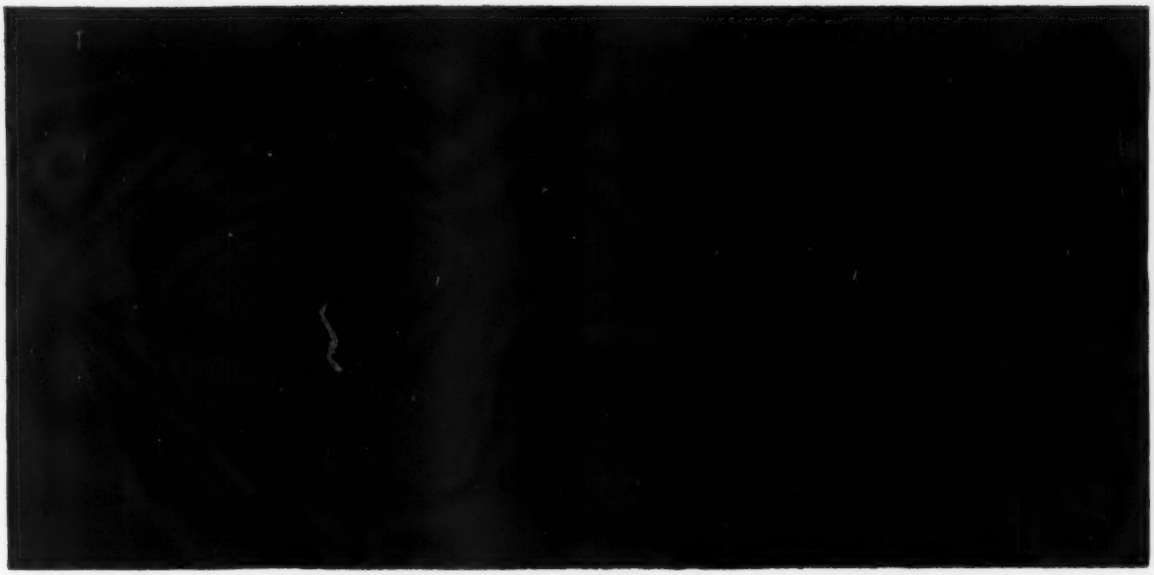

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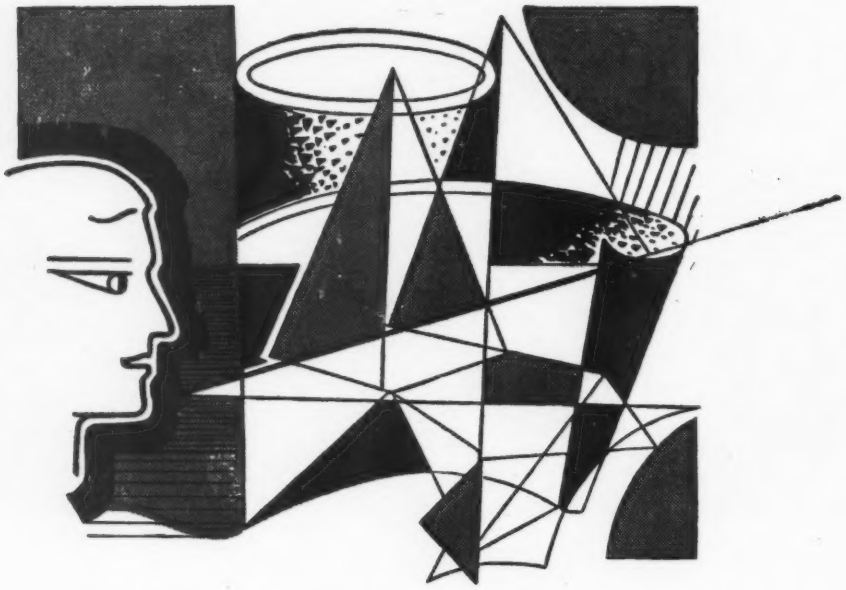

the Cylinder the Sphere and the Cone

Very probably it was the remark attributed by some to Cezanne that one "must see in nature the cylinder the sphere and the cone" which inspired Picasso in his early adventures with cubism. Cubism consists in reducing all forms in nature to their fundamental shapes. It is no longer confined to painting but has deeply influenced both sculpture and architecture.

Though as manufacturers of pipes and tubes we are apparently far removed from the field of art our interest in cubism is not limited by the fact that the pipes and tubes are cylindrical in shape, for pipes and tubes apart from serving as mediums for transmission of fluids and gases are being increasingly put to structural use both for strength and aesthetic appeal.

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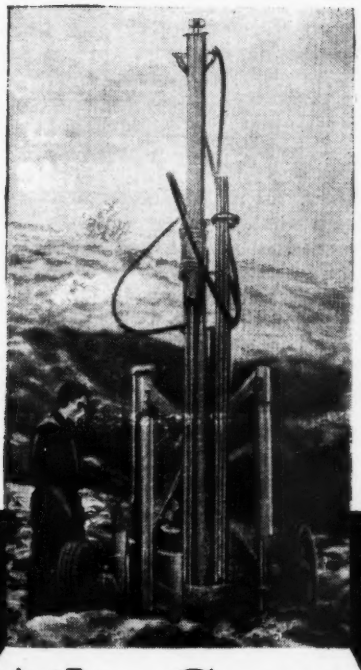
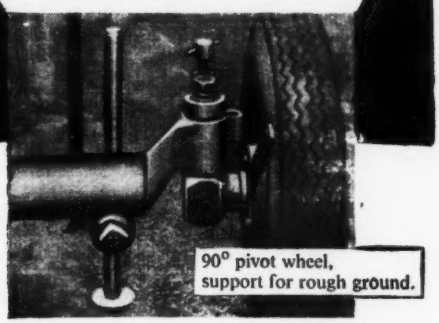


Atlas Copco Wagon Drills for precision benching



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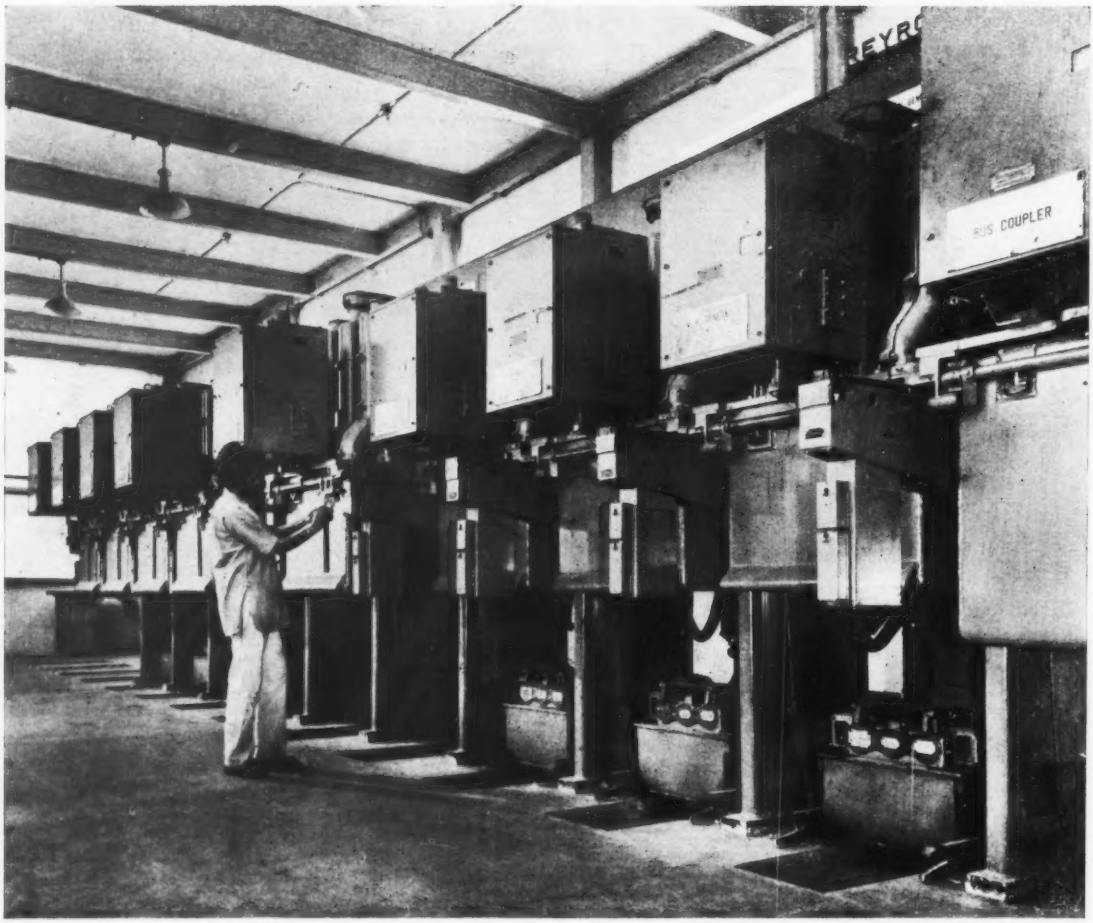

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A MONTHLY DEVOTED TO DEVELOPMENT OF POWER & WATER RESOURCES IN ASIA

Editor : K. K. Sinha

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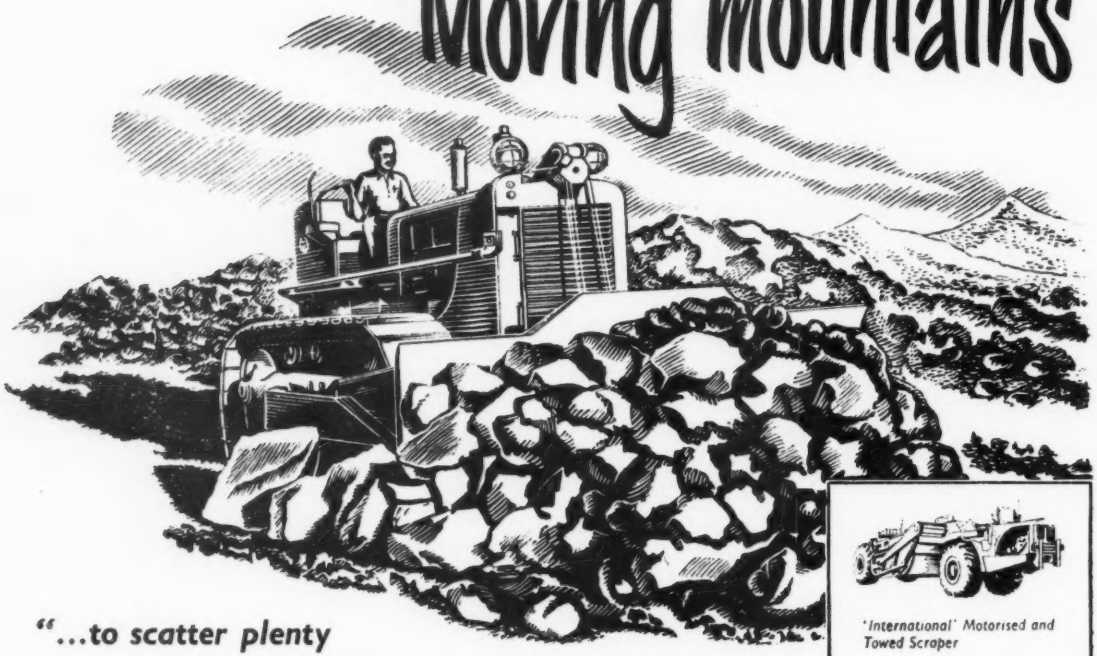

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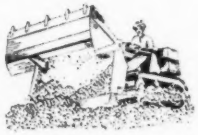

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TVA Around the World

The TVA completed twenty-five years of its life this year. Ever since its inception in the early thirties it has been an inspiration to not only the American continent but to the whole world, especially in the underdeveloped regions. It has performed its allotted tasks admirably successfully. To some it has proved itself to be too successful! We are publishing the following contribution by one who has studied several of the river valley projects in India closely. His views are thought-provoking and have an air of freshness, although we do not necessarily agree with him on all points. Sufficient experience of the TVA and other similar authorities has now accumulated in these 25 years to enable us to draw some broad conclusions. It is with this object in mind that we publish the following contribution on this anniversary year, for which we are thankful to the 'Progressive' of U.S.A.

By HENRY C. HART

"President Tito Visits 'French TVA'," "A TVA for the Khuzestan Region," "Colombia's TVA Gets the Green Light"—these are among last year's headlines from the New York *Times* and *Time* which illustrate a semantic development Julian Huxley noted 15 years ago: "In almost every country, the initials TVA stand for the multiple use of a single river to meet all the needs of man."

That the initials of an American agency should be the word by which most men everywhere refer to intensive river development is itself something of an achievement. But has TVA given more than its name to foreign projects?

What we need to answer that question is the view of foreign political leaders themselves—the men who planned and legislated the projects. We can sample that view authentically in the case of India—a nation which is not underdeveloped in recording and publishing its political decisions. We need to go back fifteen years to the genesis of India's TVA—the Damodar Valley Corporation.

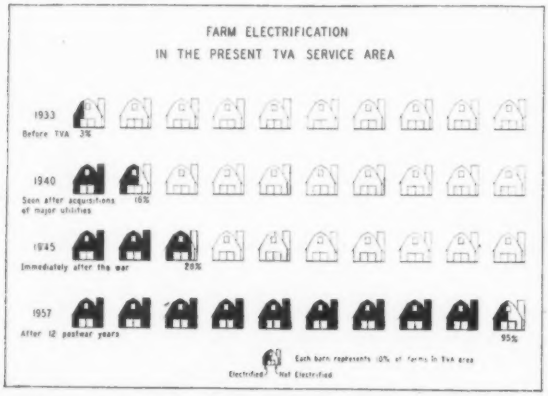
The Indian Experiment

World War II was closing in on India in July 1943 when the Damodar River burst its levees. It is not a big river, but the flood struck at a crucial time, destroying 200,000 acres of rice, cutting the rails to Calcutta, and sharply intensifying the horrible Bengal famine. The wartime British governor of Bengal was an action-minded Australian, R. G. Casey. To the Viceroy, Lord Wavell, he sent off a preliminary proposal to store the floods for hydro-electricity and irrigation. At one point, the proposal said flatly, "The nearest parallel to the Damodar is in the Tennessee Valley, U.S.A." Lord Wavell's reaction broke directly away from the normal strategy of empire. Against the recorded opposition of British interests who would lose not only prestige but probably design and equipment contracts for the project, he cabled the British ambassador in Washington to enlist a TVA engineer to come to this British colony, not on a flying consultant trip, but to stay and design the engineering harness for the Damodar.

The man who went was William L. Voorduin. In his ten years at TVA he had had the opportunity to create for himself a then new profession—that of planning engineer. His skill was in the strategy, not merely the tactics of using a river. In six months, as actual head of an Indian government agency, he had produced a plan for "The Unified Development of the Damodar River." (That phrasing of the title is the hallmark of the TVA influence on river engineering around the world.)

Two years later Voorduin left India in the midst of the bloody Hindu-Muslim riots, afraid that his foresighted scheme was lost in the birthpangs of the new nation. Meanwhile, other men, sophisticated in Indian administration and politics, had taken up the cause. Dr. Sudhir Sen, sent from his post at the Indian embassy in Moscow to Knoxville to observe the TVA model, whent home to negotiate the agreement with the two Indian States through which the Damodar flows.

In 1948, when the bill creating the Damodar Valley Corporation reached the floor of the Indian legislature, shrewd minister N. V. Gadgil prepared for the job of steering it through by watching a TVA movie. He



Progress of Rural Electrification in TVA Region

declared with pride, "The bill that is now before this House is modelled on the provisions of the Tennessee Valley Bill which was passed by the Congress of the U.S.A." The only criticism that argument drew was from an M.P. who thought a minor provision of the Indian bill departed needlessly from the language of the TVA Act! Without making a search of the record of laws transplanted among nations, I doubt whether there are many analogies to this situation. Dictators have imposed foreign models often enough, and former colonies have copied the mother country. But to search the world, and take from a country with which you have had no previous relations a pattern for an agency, an engineering plan, a form of intergovernmental relations, and a statute, is something else again.

The use India has made of the TVA example is, by now, greater still. Sudhir Sen, as Secretary of the Damodar Valley Corporation, repeatedly used TVA experience in solving his agency's problems. Prime Minister Nehru consulted two former chairmen of TVA, Arthur Morgan and David E. Lilienthal. Andrew Komora,once construction engineer at Norris Dam, has now finished his sixth year, and sixth dam, as chief engineer of the DVC. When he showed me around the half-finished Maithon Dam, speaking proudly of the young Indian engineers who had taken command, he chuckled over one curious TVA contribution. It was a large manganese-steel casting—the jaw of a heavy-duty rock crusher. After sizing the stone for Norris Dam's concrete, it was sold to Shasta in California. DVC had bought it at a bargain and it was still inexhaustible.

International Demonstration Project

What are the sources of TVA's apparently limitless influence overseas? First, certainly, its high visibility. In the first ten years after the war, TVA had 12,000 visitors from 90 ountries. Some were tourists. But others were men like the directors of the Corporacion Autonoma Regional del Cauca (Colombia); Ambassador Norman Makin of Australia, whose government had asked him to get information for the Snowy Mountains Hydro-Electric Authority; President Aleman of Mexico and his Secretary for Hydraulic Resources; the chief civil engineer of Uruguay's Rio Negro Project; Abdullah Khan, president of the Helmand Valley Authority in Afghanistan.

What these hundreds of engineers, planners, and law makers saw in TVA was a demonstration of work their countries needed. They saw how it had been done, not only the concrete and the steel, but the wording of the law, and the relaxed, man-to-man way the TVA-paid assistant country agent talked phosphates with a Cumberland Mountain farmer. The confidence of accomplished results, as well as the techniques, equipped them to win support for a TVA counterpart back home. Of all the things that are done to develop a poor country, dams, canals, and transmission lines have the greatest power to catch the eye—and the imagination. But the dramatic look of TVA has been given a limitless reach and a profound as well as a sharply-defined meaning by the articles, books, and speeches of David E. Lilienthal and Gordon Clapp, and the band of professionally competent crusaders who worked for TVA.

Role of the Alumni

The TVA'S alumni are, indeed, the second great source of its influence abroad. The roster, far too long and varied to list here, certainly begins with Lilienthal and Clapp. In 1955, after each had served as an international consultant on a highly delicate mission, Lilienthal on the Indus River which flows from India through Pakistan, and Clapp on the Jordan, they teamed up with an experienced international investment banking firm, Lazard Freres, to establish the Development and Resources Corporation. The TVA's pioneers set up shop at 50 Broadway, not many doors off Wall Street. Lilienthal put the new firm to work advising the government of Colombia how it could exploit the water and power of the Cauca River to open up for farming and industry a beautiful valley high on the western slopes of the Andes. The first dam is almost finished now; it will send 90,000 kilowatts into the booming city of Cali in the heart of the valley. Flood control, irrigation for sugar cane and rice, a new highway down to the Pacific, and new industries are coming with the dam.

The Lilienthal-Clapp firm is now deeply involved in another TVA-like enterprise in Iran. Director Abol Hassan Ebtehaj of the Iranian Plan Organization called on their corporation in 1956 to see what could be done with the barren southwest corner of his country, Khuzestan, one of the hottest places inhabited by men. When the Lilienthal-Clapp plan came in, the decisive Ebtehaj promptly commissioned their firm to build it. He released $38 million of Iran's oil money to get the project started. The Development and Resources Corporation had never before accepted construction responsibilities. But just about now dirt will begin to move on the first Khuzestan dam in a gorge of the Diz River 150 miles north of the Persian Gulf. By 1962 it will be storing water to irrigate an eventual 250,000 acres and generate 80,000 kilowatts of power.

As in TVA, the river will be made to serve a complex of agricultural and industrial developments already authorized to be built halfway down from the dam to

the Persian Gulf. A refinery will process sugar from the irrigated plantations. To this now desolate spot will come a sure water supply from the reservoirs above, electricity from the thermal plants at the Abadan refineries (even before the dam is complete), and an enormous supply of piped-in natural gas. The first industry will be a plant turning out nine million pounds of plastics per year. "From the petrochemical point of view" said Lilienthal last October, "this area resembles the Gulf Coast 25 years ago, except its resources are greater."

Coordinator of the international planning, design, and construction group in Iran is another TVA alumnus—John B. Blanford, Jr., former general manager.

•

The variety of jobs being done on river valley development abroad by other former TVA people is suggested by even a small sample :

Carl A. Bock, chief engineer, and from 1953, head of the Water Resources Authority of Puerto Rico.

The TVA Record 1933-1958

The owners of this enterprise, the people of the United States, made a substantial investment in TVA. The investment now is paying dividends in dollars and cents, in the vigour of economic life, and in its outstanding contribution to the national defense.

In the beginning, TVA was frankly called "an experiment." Today, it is a $2 billion "going concern." Resources once idle are at work. Resources once wasted are being conserved and used.

The Tennessee River System, fourth largest in the country, has been brought under control. Every basin-wide flood, including the greatest in 90 years, has been passed downstream with little harm.

An all-year navigation channel connects the great trans-Appalachian region with the inland waterway system directly connecting 20 States.

Thirty-one major dams and seven large steam plants generate electricity in quantities that dwarf the demands of America's largest cities, and more than half of it goes to atomic defense plants.

At Muscle Shoals, Alabama, the Fertilizer-Munitions Development Centre provides for the Nation's farmers and for the fertilizer industry the country's only public research organization equipped for complete development of fertilizers from laboratory to demonstration-scale production. In wartime it serves as a chemical defense plant.

In 18 States, team research in fertilizer problems has been established with the agricultural colleges. Also with college collaboration, educational programs are in operation in 34 States to bring the latest findings in fertilizer science and fertilizer use to the practical farm, as well as to fertilizer distributors and their local dealers.

Tennessee Valley farms are steadily turning from row crops to pasture agriculture. Green fields are healing the open wounds of erosion and providing feed for livestock to supplement one-crop farming.

The forests, which cover over half the Valley, are gradually being placed under scientific management by their owners. Idle, eroded areas are being reforested.

Malaria, which once sapped the strength of a third of the people along some stretches of the Tennessee, has not been detected for nine years.

Recreation, the bonus of river development, has become a major industry in which States, counties, cities, and private individuals have invested over $72 million.

Perhaps most important of all, when the security of the Nation was in peril in two terrible wars, the resources of a strategic region were mobilized quickly into the Nation's scheme of defense to produce aluminum, chemicals, and the materials for atomic weapons.

As this contribution to security and defense was national in significance, so have the region's resources been brought to the service of the Nation's commerce, agriculture, and industry.

The new navigation channel is being used on an increasing scale by shippers in many mid-western and southern States to bring oil, steel, coal, grain, and other commodities into the Valley, or to receive goods from Valley producers.

The structures regulating floods are designed and operated to help protect the lower Ohio and lower Mississippi Valleys, benefiting millions of acres of land in those areas.

Research into new fertiliser materials and processes, resulting in more than a hundred patents, has been made available without charge to scores of private industrial firms. This same type of research has helped make possible the economical development of the vast phosphate fields in western States and extended the utilization of the Nation's phosphate reserves.

Farmers in every major agricultural region are making better use of fertilizers—and of their land because of TVA-sponsored educational activities.

—From A Report to the Nation by the TVA on its first 25 years.

James B. Hayes, chief engineer of the 1948 Palestine Commission, and Van Court Hare, engineer for the 1953 U.N. Advisory Commission, to plan development of the Jordan Valley.

A. A. Meyer, chief design engineer, and William Hayes, technical adviser, of the Cauca Valley Corporation in Colombia.

John P. Ferris, in general charge, to 1957, of the work in southern Asia of the U.S. International Cooperation Administration.

What is it that commends men like these not just as consultants or contractors, but to carry some of the responsibility of the governments that hired them? I think it is the demonstration of their TVA performance that they can put fine technical skills to the service of public aims and economic transformations that reach way beyond engineering.

The Paradox

"If we are successful here," Franklin D. Roosevelt told Congress in calling for TVA, "we can march on, step by step, in a like development of other great nature territorial units within our borders." What an ironic sequel this 25th anniversary of that eloquent message reveals. At least eight nations have "marched on" consciously to apply the lessons of TVA to their rivers; the U.S. government, author of the experiment, has stood still.

I think it is possible, by examining the record of the export of the TVA idea, to come close to explaining this paradox. The TVA has been taken as a model, first, for rivers of middle size. Little watersheds are developed as features of agricultural or city water supply programs. The giants—the Indus, the Nile, the Rhine—like our own Ohio and Missouri, are too big to encompass with a unified development.

It is interesting, too, that the TVA idea has been most usable in semi-developed nations. To take the first steps from poverty and tradition, a single purpose development may stretch a nation's capacity—the old British Gezira irrigation scheme on the Sudanese Nile may seem a more appropriate model than TVA. It was a brutal aspect of the proverty of India when she attained independence that only in one small valley, the Damodar, could she afford a plan to stop floods. Her larger rivers had to be harnessed initially for irrigation and electricity, which would pay out in money, not lives. The highly developed countries of western Europe, and even Japan, have found their river valleys already committed to earlier navigation or hydro-electric development, or water supply, or irrigation, or transport, or industrial uses too far-reaching to incorporate into a unified plan centered on the river. The TVA idea has fit best the hitherto slightly exploited river basins in nations (or even empires), having some means, some markets and some qualified men, but wanting more.

So far, too no country has built a *second* project of TVA pattern. Perhaps the intensive development of a river, apace with the related development of forests, farms, and minerals which can be tied in with it via hydro-electricity, or transport, or irrigation, or watershed protection, is inherently a pilot plant operation. Perhaps, as some entirely dispassionate American political scientists believe, the tie of the land and the minerals to the river that drains them is not made by nature, but by imaginative men. And once the value of the development of related resources has been demonstrated, nationwide programs tend to spring up—soil conservation, encouragement of small industries, rural electrification, adult education, and many more. The valley authority may have extended its lessons without extending its form.

Influence of the World Bank

Certainly there is one powerful force working for valley authorities abroad which is missing at home—the World Bank. The utterly pragmatic loan examiners sent out by Bank Chairman Eugene R. Black want irrigation, and kilowatts, and increased productive capacity for every dollar they invest. Projects for hydro-electricity, irrigation, and flood control, incidentally, have received more loans from the Bank (close to one billion dollars) than any other type of project. And so, as for the Damodar, the Cauca, the Ping River in Thailand, the Litani River in Lebanon, the Bank itself has insisted on making its loan to a regional, businesslike agency—an agency like TVA. To quote Black's reasoning:

> "The Bank places particular stress upon the assurance of adequate management for the project.... In this connection, the Bank has frequently insisted, in the case of government projects, that their operation be entrusted to a quasi-autonomous authority, or in some other fashion be insulated from political pressures and the rigidities of government administrative procedures."

TVA's around the World

Projects expressly modeled in part on TVA include:

Damodar Valley Corporation	India
Helmand Valley Authority	Afghanistan
Khuzestan Project	Iran
Puerto Rico Water Resources Authority	Puerto Rico
Corporacion Autonoma Regional Del Cauca	Colombia
Snowy Mountains Hydro-Electric Authority	Australia
Papaloapan Commission	Mexico
Litani River Authority	Lebanon

Continued on page 34

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Design Criteria for Power Tunnels in Rock

By H. D. SHARMA

Design of power tunnels in rocks comprises the following steps :—

(1) Determination of tunnel diameter and grade.

(2) Assessment of rock loads likely to occur due to roof yield and provision of adequate lining.

(3) Design of tunnel lining to withstand internal pressure head.

(4) Provision of drainage arrangements to relieve external hydrostatic pressures and elimination of seepage in vicinity of important structures.

These points will be dealt with in detail as below:—

Diameter and Grade

The tunnel diameter is primarily decided on basis of economics and is a problem of balancing between revenue loss due to tunnel friction, interest charges on initial capital cost of tunnel, annual maintenance cost and capitalized abatement. The sum of all these costs must be a minimum for the most economical size of tunnel diameter. To establish a mathematical relation between the above costs, let:

Q = Discharge in conduit, in cusecs,
N = Coefficient of rugosity of conduit surface,
d = Diameter of conduit in ft.,
R = Selling rate in Rs. of power per k.w.h.

From Manning's formula and with 90% turbine efficiency, the annual power loss per running ft. of tunnel works out to :

$$= 3140\ N^2 Q^3 d^{-5.33}$$

$$\text{Annual Revenue loss per rft.} = 3140\ N^2 Q^3 d^{-5.33}.\ R = K_1 d^{-5.33}$$

Total cost of excavation and lining, with $7\frac{1}{2}$% overbreak and assuming lining thickness to be d/12ft, works out to:

$$= (1.15 d^2 C_e + 0.364\ C_L d^2) \text{ per rft.}$$

where C_e = Cost of tunnel excavation in Rs. per c. ft.

C_L = Cost of concrete lining in Rs. per c. ft.

Allowing interest, amortization, maintenance etc. @ p% annual operational expenditure on tunnel :

$$= p.\ d^2 (0.0115\ C_e + \cdot 00364\ C_L) \text{ per rft} = K_2 d^2.$$

Adding and equating differential coefficient with respect to 'd' to zero, we get:

$$d = \sqrt[7.33]{2.67\ \frac{K_1}{K_2}}$$

$$= \left[\frac{2.67 \times 3140 \times N^2 Q^3 R}{p\ (0{\cdot}0115\ C_e + 0{\cdot}00364 C_L)} \right]^{0.137} \quad \ldots\ldots\ldots \text{(i)}$$

Other considerations for fixation of diameter size are the space reqirements for equipment proposed to be used in tunnelling, time factor, difficulties encountered in excavation of rock and availability of construction machinery. The tunnel diameter determined on basis of economic formula must be examined from practical considerations mentioned above and modified if necessary. Normally for tunnels less than 20 ft. diameter, it becomes difficult to work with mechanical equipment due to cramped working space inside. Where steel liners are used, practical maximum sizes are 18′ to 22′ dia. based on considerations of available plate thickness, manufacture and handling. A size larger than this is not so commonly used except under special circumstances.

The maximum tunnel size is also limited by the type of rock strata through which it is to be excavated. Tunnels upto 70 ft. diameter have been constructed in granite in Norway. Where the rock is poor, the roof loads for a large diameter tunnel may result in prohibitive cost of tunnel lining and increase of hazard in actual construction and subsequent maintenance.

Normally, from point of view of minimum power losses, one tunnel is aways to be preferred over two or more tunnels as it will give least friction losses for the same equivalent area of cross-section. Where, however, the magnitude of discharge to be handled requires an excessive value of tunnel diameter for the

particular types of rock strata, it my be advisable to provide two or more tunnels whose sizes can be determined by the economic formula already described.

The two factors controlling tunnel grade are:—

(1) Upstream level of tunnel at the forebay. This is fixed such as to provide adequate seal over the soffit of unnel entrance under conditions of maximum drawdown. Considerations of minimum silt entry into the tunnel require high setting of intake, the final level depending on judicious provision for both.

(2) Tunnel exit or end level is essentially determined by considerations of adequate seal over the tunnel soffit for lowest depth of surge at the time of most critical load demand during turbine operation. This seal is required to guard against entry of air into the plant which may result in severe vibrations of turbine, hunting and undesirable load variation. The surge depth has to be computed for extreme conditions that may arise at any time due to most unexpected sequence of turbine operation. Values of surge depths must also be worked out after allowing choking effect in supply conduit due to increased value of rugosity co-efficient as a result of prolonged incrustation of pipe surface.

Rock Loads for Tunnel Lining

The construction of an opening in a rock disturbs its equilibrium and causes the adjacent rock to move into it. This tendency is counteracted by developing self-balanced system of internal stresses that produce the phenomenon known as arching around the tunnel. If the rock is good and sound, the entire vertical load will be taken up by arching, and no tunnel lining will be necessary. If the rock is weak, a part of overburden will be taken up by lining. In order that shearing stresses in rock around the tunnel may develop, small displacements must occur in the overburden. Hence, if a tunnel is lined, the lining should be flexible enough to permit small displacements and mobilise shearing stresses. If the lining is too flexible, the integrity of rock mass would be destroyed, the shearing stresses would vanish and the lining would be loaded with whole load of overburden. Though the requirements are clear and simple qualitatively, a quantitative solution of lining problem does not exist and lining design is mostly based on practice and engineering judgement.

In all tunnels in more or less homogeneous rock, the material tends to cave in. As already pointed out, if the rock is capable of developing sufficient shear stresses the tunnel will be stable without the necessity of lining. In fissured or stratified rocks, however, the shear stresses are only poorly developed and rock fall in roof always occurs. In some cases, this is not apparent just after construction, but may gradually develop due to creep effect resulting in collapse of tunnel. Practice and observation show that the vertical pressure on the roof of a tunnel in the rock is generally independent of depth. In other words the shape and magnitude of the caving zone are the same for all depths of tunnel in a particular rock.

For convenience in design computations, the rock load is assumed as an equivalent uniformly distributed load over the tunnel roof with span equal to tunnel diameter. Empirical values for all types of rocks have been set forth by K. Terzaghi and are reproduced as below :

Rock load in ft. at a depth of more than 1.5 $(B+H)$

1	Moderately blocky and seamy rock	0.25 B to 0.35	(B+H)
2.	Very blocky and seamy	0.35 to 1.1	(B+H)
3.	Completely crushed but chemically intact	1.1	(B+H)
4.	Squeezing rock moderate depths	1.1 to 2.1	(B+H)
5.	Squeezing rock, great depths	2.1 to 4.5	(B+H)
6.	Swelling rock	up to 250 ft.	

Where B and H represent width and height of tunnel as indicated in the figure.

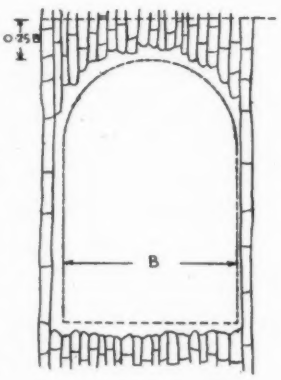

FIG. 1.

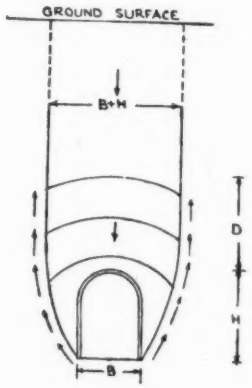

FIG. 2.

The roof of the tunnel in the case of first three ground materials listed in the table is assumed to be located below water table. If it is located permanently above water table the values given for those materials can be reduced by 50%

These values are based on the consideration that the lining in stratified or moderately jointed rock has to support only some loose rock due to gradual readjustment in the state of stress and failing of interlocking action between adjoining rock fragments dut to creep, and its height seldom exceeds 0.25B. Even this value is the ultimate value of roof load that is likely to develop if the rock is not properly wedged against tunnel supports during construction. In other cohesive rocks, the actual caving body is replaced by a conventional ground arch of variable width as shown in fig. 2. The width of ground arch depends on character of the rock and is roughly proportional to the sum of (B+H). This is based on the

assumption that the shear planes of active earth pressure rise at an angle of approximately $\left(\frac{\pi}{4}-\frac{\Phi}{2}\right)$ with the vertical and mark the boundary of zone of arching within which the entire rock mass has a tendency to become loose after excavation of tunnel. The value of thickness of uniformly distributed equivalent ultimate rock load will be given by K (B+H) in such cases, the value of 'K' being given in the above table. With loads determined as above, the tunnel be designed like an ordinary concrete pipe.

Elastic Stresses in Rocks

Normally all rocks are elastic within certain limits, though strictly speaking their modulus of elasticity may not be the same in all directions. The smallest value of E generally occurs when it is measured normal to the bedding planes of a rock. In a general case, the elastic properties of a rock are adversely affected by various geological conditions such as fault zones, fractures, small intrusions in sedimentary rocks and lenticularity of bedding which permits the intermission of soft zones into generally hard rock. Nevertheless, a mean value of E may always be arrived for all types of rocks to correspond to rock stresses. Roof grouting in tunnels generally improves rock condition and actual stress distribution may, therefore, be fairly close to theoretical values.

A tunnel in rock may be taken as identical with the case of opening in a semi-infinite plate subject to loading in one or more directions. The values of peripheral stress for an isotropic plate subjected to unidirectional compressive stresses are given as below :— (Vide Theory of Elasticity by Timoshenko.)

$$\text{Radial stress} = p_r = \frac{p}{2}\left(1-\frac{a^2}{r^2}\right)+$$

$$\frac{p}{2}\left(1+\frac{3a^4}{r^4}-4\frac{a^2}{r^2}\right)\cos 2\Theta \ldots \text{(ii)}$$

$$\text{Tangential stress} = p_t = \frac{p}{2}\left(1+\frac{a^2}{r^2}\right)-$$

$$\frac{p}{2}\left(1+\frac{3a^4}{r^4}\right)\cos 2\Theta \ldots \text{(iii)}$$

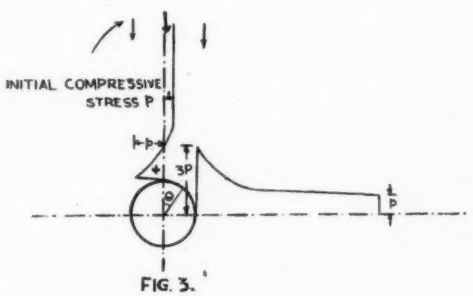

FIG. 3.

Where 'Θ' is angular distance of point in question from soffit;

'a' is radius of circular hole;

'r' is radius of distance of point where stresses are required;

'p' is unidirectional compressive stress acting vertically. These stresses are illustrated in figure showing their distribution along and in the vicinity of periphery.

It will be seen that the tangential stresses at the soffit, and sides for load applied in vertical direction are -P and 3P just at the periphery and go on gradually decreasing till they become approximately equal to normal stress within an approximate distance equal to radius of the hole.

If a system of horizontal stress is superimposed on the plate, the equation for tangential stress takes the shape:—

$$p_t = \left[\frac{p}{2}\left(1+\frac{a^2}{r^2}\right)-\frac{p}{2}\left(1+\frac{3a^4}{r^4}\right)\cos 2\Theta\right]+$$

$$N\left[\frac{p}{2}\left(1+\frac{a^2}{r^2}\right)+\frac{p}{2}\left(1+3\frac{a^4}{r^4}\right)\cos 2\Theta\right] \ldots\ldots \text{(iv)}$$

$$\text{Where } N = \frac{\text{horizontal stress}}{\text{vertical stress}}$$

The formula may be used to give the approximate state of stress within rocks after tunnelling. The following figures indicate the stress distribution after excavation of tunnel for values of N=1, and $\frac{1}{3}$.

Terzaghi and Richart in their theoretical treatment of stresses around cavities in rocks have shown that the vertical stresses in a rock mass at a depth Z below its surface are given by:—

$$P_v = wz \ldots\ldots\ldots\ldots\ldots\ldots \text{(v)}$$

where w is the unit weight of rock.

If the rock were free to expand laterally, no other stresses except the vertical would exist in the rock. However, the lateral expansion of rock is prevented by adjacent rock masses and horizontal stresses are invariably induced in rocks, depending on the homogeneity of the rock. These stresses may be indicated by the equation:—

$$P_h = N.wz = N\,P_v \ldots\ldots\ldots\ldots \text{(vi)}$$

N being a dimensionless number indicating ratio of the two stresses.

Derivation of the theoretical value of N will make its concept clear. Consider a prism with a unit weight 'w' placed on a perfectly frictionless base. The pressure due to weight of the material causes not only a vertical deformation but also a lateral expansion.

The magnitude of lateral expansion at depth Z is given by,

$$e = m.\frac{wz}{E} \quad \ldots\ldots\ldots\ldots\ldots \text{(vii)}$$

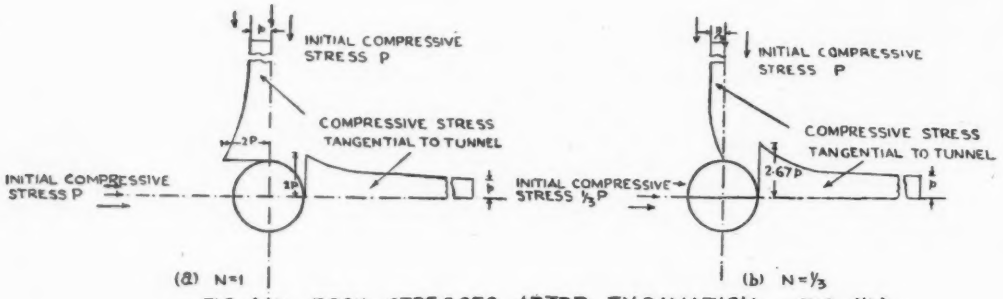

FIG. 4 (a) ROCK STRESSES AFTER EXCAVATION FIG. 4(b)

E = Young's modulus of the rock, and m = Poission's ratio.

If the prism is laterally confined between perfectly smooth vertical walls or within a layer of identical material on rough base, no lateral expansion can occur. Therefore, at any depth, every vertical side of the prism will be acted by a horizontal pressure such that it it reduces the lateral expansion to zero.

The lateral deformation due to the horizontal pressure:

$$e' = N. \frac{wz}{E}(1-m)$$

Equating this with original expansion as per equation (vii)

This gives $N = \frac{m}{1-m}$ .....(ix)

With a value of 0.25 for Poission's ratio, N=1/3

For practically incompressible materials, the value of 'm' = 0 .5 and N in that case works out to unity.

For rocks deposited in layers in geologically undisturbed regions Terzaghi and Richart have given values of N betwee 0.11 and 0.43. Measurements by US.B.R. indicated value of N to be 0.6 in case of Gorge extension tunnel and as much as 3 for Boulder Dam tunnels, high value being due to structure being located beneath a preloaded steep-sided valley. On the other hand European investigators suggest that rock at depth can be considered as a plastic material and N can be taken approximately equal to unity

Due to wide range over which the value of N may vary, it is always best to make observations for individual cases. This, however, is not possible due to time limitations and structures have to be designed on the basis of other similar works constructed before.

With a value of N=1 in the expression of tangential stress, it works out to 2 wh throughout the periphey of tunnel. This implies that when a tunnel is excavated through rock, a redistribution of stresses takes place making the radial stresses zero and tangential stress equal to twice the vertical., Since these stresses are compressive, the tunnel stands without any support. Equating maximum allowable compressive stress of rock to 2 wh, it can be shown that limiting depths for tunnel excavation are about 35,000 ft. for granite and other igneous rocks, and about 5000 ft. for sandstone.

Rock Stresses Due to Internal Pressure In Tunnel

When an internal pressure is applied in a circular tunnel radial compressive stresses and tensile stresses are induced at the inside surface equal in magnitude to the internal pressure. The distribution of these stresses in the rock mass around the tunnel can be worked out by the theory of stress distribution in a thick walled cylinder. The tangential and radial stresses in a thick cylinder are given by Lame's equations:

$$p_t = \frac{a^2.p_i}{b^2-a^2}\left(1+\frac{b^2}{r^2}\right) \text{.....(x)}$$

$$p_r = \frac{a^2.p_i}{b^2-a^2}\left(1-\frac{b^2}{r^2}\right) \text{.....(xi)}$$

where, p_t and p_r denote tangential and radial stresses, a, b, are inside and outside radii respectively.
'r' is the radial distance of any point within the mass at which stresses are computed.
p_i denotes internal stress to which the cylindrical shell is subjected.

When 'b' i.e. the outside radius is several times the inside radius 'a', these equations tend to become:

$$p_t = \frac{a^2}{r^2}.p \text{ and } p_r = -\frac{a^2}{r^2}.p_i \text{ .....(xii)}$$

These equations may be applied for determination of stress distribution due to internal pressure within the rock mass around the tunnel. The net stresses in the rock will be the sum of the two stress systems i.e. those due to rock mass alone as described previously in equation (iv) plus stresses due to internal pressure in equation (xii).

A comparison of the two stresses will show that in a rock tunnel with N=1, it can safely stand a pressure head of p = wz without reducing tangential stresses

below pre-excavation stresses in the tunnel. Under such conditions, the factor of safety against development of tensile stresses works out to 2. With a unit rock weight of 160 lbs., and a safety factor of 2, the minimum rock cover required equals $\frac{62.5 \times H}{2 \times 160} = 0.195$ H for a water head of 'H'ft. However, where value of N falls below 1, tensile stress with consequent risk of leakage may occur earlier. For N= 1/3, cracks will start as soon as pressure is applied and will penetrate deep into the rock when internal pressure exceeds $\frac{1}{3}$ wz.

Because of uncertainty which exists with regard to the rock properties, it is generally considered that a rock cover over a pressure conduit which is not steel lined should never be less than 50% of the maximum designed internal pressure head. With this cover cracks in rock, above soffit will not occur upto a value of N = 0.46. If N is less, cracks and leakage may occur, but the rock will still provide a factor of safety of 1.28 against heaving. Under such circumstances, the provision of liner is essential.

The elastic properties of concrete lining and rock surrounding the tunnel are usually the same and if there is intimate contact between the two, they will behave monolithically and the concepts developed above will apply with sufficient accuracy. The reinforcement required in the lining against internal pressure may be computed by the above equations.

As mentioned earlier, the lining should also be designed for possible rock fall as per values recommended by Terzaghi. This is necessary in tunnels where due to low values of N, rock may develop cracks at the crown and may eventually become loose due to leakage of water into it from the tunnel. Moreover, soffit rock generally gets shattered due to use of explosives in excavation and there are strong possibilities that some cavities may remain inspite of grouting, resulting in rock fall subsequently and blockade of tunnel.

Provision of Steel Liner in Pressure Tunnels

There are a number of factors which influence the decision for provision of a steel liner in a pressure tunnel, the most important being to protect power house and other important structures from a possible hill slide or rock movement caused as a result of reduced stability factor due to hydrostatic pressures in bedding planes of rock or to prevent excessive leakage from the tunnel at points of low rock cover against heave. This becomes all the more serious in faulted, shattered, blocky and seamy type of rocks. Thus steel lining is used in pressure conduits not so much for structural strength as for providing a water tight membrane against leakage. Davis recommends that a liner may usually be provided in ordinary type of rock where heads involved are higher than 100 ft.

The thickness of steel lining is small in relation to diameter of pressure conduit and its design is determined both from consideration of safety against internal hydrostatic pressure and buckling due to external pressure.

Regarding design against internal pressure, the basic criterion is that the radial deformation of liner minus any gap between liner and concrete (Since the steel surface is generally never in intimate contact with concrete lining due to smoothness of its surface and there is always some clear gap left between the two, usually taken 1/16"—1/8". Even if there is no gap initially, it may develop later on due to reversal of stresses and plastic yield) will be equal to combined radial deformation of concrete lining, cracked rock and rock beyond cracked zone.

Let the following notations represent:—

E_s, E_c, E_r, = Young's moduli for steel, concrete and rock respectively.

b,c,d, radii inside of liner, outside of concrete lining, and to the end of radial fissure in rock respectively.

t = thickness of steel liner.
g= gap between steel liner and concrete.
m = Poission's ratio for rock.
P = Pressure in conduit
k= proportion of pressure taken outside the liner.

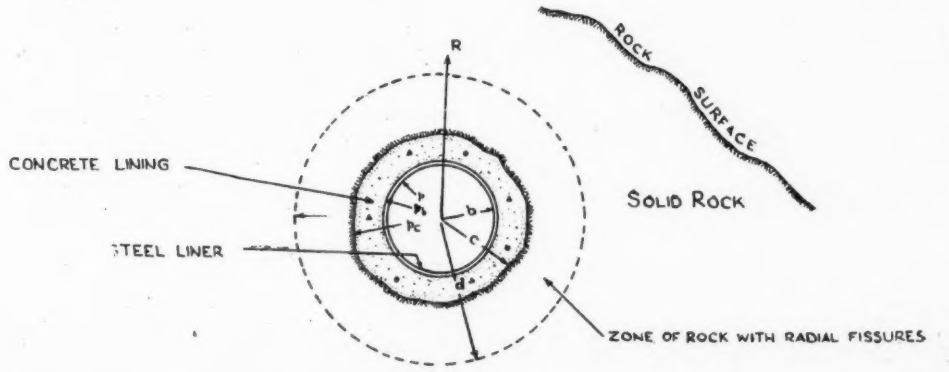

FIG. 5

Deformation in steel liner is given by:—

$$U_s = P(1-K)\frac{b^2}{E.t} \quad .. \quad .. \quad .. \quad .. \quad (xiii)$$

Deformation in concrete lining. Uc, may be computed as below: pressure at the inner periphery of lining = KP. pressure at a radius of 'r' within the lining mass = $K.P.\frac{b}{r}$

(Linear distribution of radial stress assumed since hoop stresses in concrete lining will be zero due to development of tensile cracks)

$$\text{Deformation, } U_c = \int_{r=b}^{r=c} K.P.\frac{b}{r}.\frac{d_r}{E_c}$$

$$= \frac{K P b.}{E_c}\log_e\frac{c}{b} \quad .. \quad .. \quad .. \quad .. \quad (xiv)$$

Similarly,
Deformation in fissured rock,

$$U_r = \int_{r=c}^{r=d} K P.\frac{b}{r}.\frac{d_r}{E_r} = \frac{K P. b}{E_r}\log_e\frac{d}{c} \ldots . (xv)$$

Deformation in sound rock:—

$$\text{Unit strain in circular ring} = \left(\frac{p_r}{E} - m\frac{P_t}{E}\right)$$

$$\text{Deformation in rock} = \int_{r=d}^{r=inf.} \left(\frac{P_r}{E_r} - m\frac{P_t}{E_r}\right) dr$$

Stress at the inner periphery of the rock $= KP.\frac{b}{d}$.

From equation (xii), $p_t = KP.\frac{b}{d}.\frac{d^2}{r^2} = KP\frac{bd}{r^2} = -p_r$

Substituting in equation for deformation, we have

$$U_r' = \int_{r=d}^{r=inf.} -Kp\frac{bd}{r^2}\frac{(1+m)}{E_r}dr = -\frac{Kpb}{E_r}(1+m)\,d\left[\frac{1}{-d}\right]$$

$$= \frac{Kpb(1+m)}{E_r} \ldots . (xvi)$$

Equating $U_s - g = U_c + U_r + U_r'$, we have

$$K = \frac{\left(\frac{b^2}{Et} - \frac{g}{p}\right)}{\frac{b^2}{Et} + \frac{b}{E_r}(1+m) + \frac{b}{E_r}\log_e\frac{d}{c} + \frac{b}{E_c}\log_e\frac{c}{b}} \quad \ldots\ldots\ldots (xvii)$$

Value of 'd' i.e. radial distance upto the end of fissured rock may be computed by trial and error between the above equation for 'K' and that for tangential stresses in rock around tunnel mentioned earlier, in equation (iv). This may be done by equating value of P_t from equation (v) i.e. tangential stress to Kp. b/d. i.e. the value of peripheral stress used for computing deformation in sound rock. So much effort is, however, rarely justified since even reasonably approximate value of 'd' will give satisfactory results.

It will be seen from the above equation that the gap between steel liner and concrete plays a significant role in determining liner stresses. Efforts are to be made during construction to minimise this gap, though it is not possible to completely eliminate it. This gap may widen after construction due to:

(1) Rise in steel liner temperature during setting of concrete and subsequent cooling.

(2) When the conduit is filled with water under high pressure, the shrinkage gap will be closed and pressure exerted on surrounding rock resulting in formation of radial cracks. After dewatering, the resulting gap may be larger than the original gap, as part of the deformation will be plastic and therefore non-reversible.

This gap may be kept within desirable limits by:

(*i*) Thorough grouting of outside rock to minimise its deformation.

(*ii*) Where constructional requirements permit, the gap may be reduced by dewatering after first fill up to full hydrostatic pressure and regrouting outside the liner. Such a procedure may not be possible where relief drains are provided outside.

(*iii*) By welding to the outside surface of liner, a large number of steel anchors to support the steel liner before concrete is placed between liner and sourrounding rock. This will also tend to increase factor of safety against buckling due to external load and thickness of liner can be reduced accordingly.

Buckling due to External Pressure

Since the steel liner is never in intimate contact with concrete lining it cannot be relied upon to share any flexural stresses due to rock overall loads. In good sound rocks where the tunnel roof is self supporting, concrete lining is provided not so much for structural reasons as for smoothening surface and filling up irregularities, and the question of sharing of flexural stresses by steel liner does not arise. In cases, where rock is weak and roof loads are anticipated, tunnel lining is to be designed for such loads without any reliance on and independent of steel liner.

After hydroelectric tunnel is put in operation, it is subjected to hydrostatic pressure all along its length. Since no concrete or rock is completely impervious, this pressure may be transmitted long distances through small rock fissures or through pores of concrete. It

must therefore, be assumed that since the concrete lining of tunnels and shafts is somewhere in direct contact with water, for example with water in surge tank, full hydrostatic pressures may be operating behind the concrete lining. In addition hydrostatic pressures may develop due to ground water from extraneous sources by transmission through rock seams. No effective solution has yet been developed to safeguard against these pressures by suitable drainage. The steel liners have therefore, to be designed against buckling due to external water pressures.

Analytical method for determining critical buckling stress of steel liner inside tunnel has been developed by E. W. Vaughan with the help of equations for elastic stability of a thin shell given by Timoshenko. Such a shell will deform into wave form under an external pressure and will give rise to flexural stresses in addition to direct compression, just like the case of strut in compression. Bending stress may be determined by assuming that the maximum radial deflection represents the eccentricity of direct compression. By assuming further that failure will occur when the combined stress exceeds the yield stress, Vaughan develops the following equation: (see origianl paper for details)

$$\left[\frac{p_y - p_{cr}}{2E'} + \frac{6\,p_{cr}}{p_y - p_{cr}}\left(\frac{y_o}{R} + \frac{p_{cr}}{E'}\right)\right]\frac{R^2}{T^2} - \frac{R}{T} + \frac{p_y - p_{cr}}{24\,p_{cr}} = 0$$

where p_y = yield stress of steel,
p_{cr} = critical buckling stress in the shell,
E = modulus of Elasticity for steel p-s.i.
m = Poissions ratio,
R = radius of steel shell in inches ;
T = thickness of steel section in inches ;
Y_o = deformation due to prestressing, temperature effects, and out-of-roundness of liner due to manufactural defects.

The above equation has been solved by Vaughan in the form of a graph by plotting critical external pressurs in feet of water as ordinate and values of $\frac{R}{T}$ as abcissa. Curves have been drawn for values of $\frac{Y_o}{R}$, ranging from zero to 1×10^{-2}.

Other methods of strengthening steel liner against buckling is to provide suitably spaced anchors at the outer periphery of the liner. Empirical formulae for allowing for the effect of anchorages have been determined by Dubas (vide Jaegar's paper under references), and such anchors have been provided in some projects. However, the modern practice is to dispense with the use of anchors due to following reasons:

(*i*) Any type of stiffening device attached to the lining increase the clearance for placement of concrete and may thus necessitate a larger working space than would otherwise be necessary. In addition, it becomes more or less impossible to fill the entire space completely with concrete.

(*ii*) The rigidity of anchors does not permit them to stretch sufficiently to follow the radial shortening of the lining when it is subjected to grouting pressure resulting in breakage.

Provisions for Drainage

Drainage provision in tunnels is necessitated for two reasons:—

(1) Release of hydrostatic pressures applied on tunnel lining through seams and rock fissures due to extraneous sources located inside or on hill surface. If unprovided for, their extent and magnitude may be unlimited depending on location of source and may cause complete or partial collapse of tunnel lining.

(2) The best concrete lining develops cracks due to internal pressure head resulting in water seepage into the adjacent rock. In this way external water pressures are built up around the tunnel and if the flowing water in the tunnel is suddenly withdrawn in emergency, or for maintenance reasons, the unbalaced outside water pressure may cause collapse of lining if not provided for.

Full provision for drainage in a tunnel system, therefore, constitutes a very important aspect of design. Unfortunately, no really effective solution has yet been evolved for the problem. One alternative for relieving these pressures, which has been tried in the past is to place drainage pipes parallel to the tunnel. However, past experience has shown that this arrangement has not proved satisfactory and modern views of designers are against any drainage system with pipes or galleries parallel to the main pressure tunnel due to following reasons.

(1) If drainage pipes are small running just beneath the steel lining, they get sealed when pressure grouting is applied and thus become ineffective.

(2) If drainage pipes are large, designed as inspecting galleries, experience has proved that the stress distribution on the cirular tunnel lining is no longer symmetrical. Excessive bending moments will be developed in the lining and may ultimately cause a disastrous burst.

In his paper 'Present Trends in Design of Pressure Tunnel' Jaegar mentions:

"If small drainage pipes are provided, they shall not be considered as relieving the water pressure in the rock, even though some of them keep working and spouting clear water years after the rock has been grouted solid."

Other methods of drainage which have been tried with varying degree of success are:—

(1) Attempts have been made to minimise cracking by injecting bentonite slurry behind the lining. The resultant expansion of bentonite places the lining in compression reducing thereby and also minimising leakage into the rock due to its impermeability.

[*Continued on page* 17

THE BEST IS YET TO BE

Landmarks as they are, Bhakra Nangal, Damodar Valley & Hirakud are but a beginning—beginning of the onward March towards better living and prosperity. Today steals a march over yesterday only to be bettered by the tomorrows—bettered by the experience and confidence derived by our dedication. For the best will be when India meets the Great Challenge.

KAMANIS starting from the scratch, can now pride themselves in having a most up to date factory where design, testing fabrication, and galvanising of Transmission Line Towers are all done under the same roof. Since the inception they have designed 184 types of Towers, 92 types of sub-station structures and 28 types Communication and Aerial masts. Orders executed by KAMANIS and orders on hand aggregate to 45,000 tons. This is but a beginning and the best is yet to be.

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Efficiency of Construction Equipments

T. P. MARIK

In the past we depended entirely upon manual labour and ordinary or improvised construction equipments and I believe the then situation was tackled satisfactorily. But the situation has changed considerably and is changing fast and unless we deal with the problem expeditiously and with proper understanding and evaluation of all factors concerning the building industry, I am afraid, we shall fail to abide with the time and will lag behind.

It is true in our populated country there is no dearth of ordinary labour but we do not possess sufficient strength of skilled and trained labour required to build the engineering projects of modern and advanced specifications. Further, modern specifications of works, involving extensive use of cement and steel, command production of high grade concrete and fabrication, the most important factors contributing to the strength of the works, and this can only be achieved economically with the help of modern construction equipments.

On the other hand, when the wages of labour in our country are very much cheaper compared to those in many developed countries, we should not try to make our industry fully mechanised but should utilize the available labour to the greatest extent possible without impairing the strength and efficiency of the construction.

In order to execute the works economically we must make correct apportionment of the labour to construction plants in carrying out various operations, so as to ensure that both the labour and the plant are kept actively employed without any appreciable idling loss.

I need not deal further with this subject as it is outside the scope of my charge. I shall now deal with some of the modern equipments that are being commonly used at present in my part of the country in modern construction projects. I shall discuss mainly about the output and efficiency of these equipments. A short description of these equipments and their utility is given below.

Crushers and Granulators

These are used to crush and reduce stone boulders down to $\frac{3}{4}$ in. dimensions which are commonly used as coarse aggregates in cement concrete. Crushers are used to crush stone boulders down to approximately 2 inches.

Granulators are used to reduce stone of approximately 2 inches dimensions to finer limits down to $\frac{3}{4}$ in.

To cope with the demand of coarse aggregates in huge quantity for concrete which is very popular now and is essential in respect of many important structures, the old system of breaking stone by manual labour has to be supplemented, wherever the situation so demands, by installing crushers and granulators.

Concrete-Mixing Plant

There are two types of Mixing Plants viz :—

(1) The revolving-drum Type

(2) The Continuous-mix Type

Of these the former is most commonly used in this country. The revolving-drum type of mixer is used with either a tilting drum of conical shape which rotates on a tilting axis of a closed or non-tilting drum which rotates on rollers.

In producing concrete the use of a concrete mixer is desirable from many points of view. Mixing of concrete in a plant is not only more economical than mixing it by hand but the output is much more too. Further, it ensures thorough mixing of aggregates with cement and controlling of the water/cement ratio, two most important factors in producing high grade concrete, which can hardly be achieved by hand mixing.

Further, where voluminous work is to be carried out requiring continuous mixing, the system of mixing

concrete by hand will not operate smoothly due to paucity of trained labour. Producing concrete in a mixing plant requires much less trained labour and so, whatever trained labour is available in the region may be utilised in carrying out other works.

Vibrators

(1) *External Vibrators*—These are particularly suited for compacting in-situ work in shuttering, tunnel lining work, road making etc. These are also used to vibrate moulds.

(2) *Internal Vibrators*—These are used to compact concrete in structures. The vibrator needle is immersed in the concrete and worked up and down in a stirring motion.

Modern specifications call for concrete of high quality, both in respect of precast units and concrete in-situ work. This requires, besides careful selection grading and proportioning of the aggregates, a low water-cement ratio, so important and essential for producing high grade concrete at a cheaper cost. Vibrators make possible the use of this much drier concrete which could not normally be effectively compacted by hand.

By using vibrated concrete, we get the following advantages :—

(1) Increased compressive and flexual strength;

(2) Better bonding to reinforcement involved;

(3) Increased density and resistance to absorption;

(4) Better weathering properties.

Elevators or Hoists

Various types of hoists are used in the building industry, the type and size suited for a specific operation depending upon :

(1) the nature of work on which it is to operate ;

(2) the load it has to handle and lift.

The type of hoist used in the work is known as "Hoistex". It comprises mainly of the following :

(1) Hoist (Winch)—It can be either stationary or used as a mobile unit with wheels, having diesel or petrol engine or electric motor drive;

(2) Tower—Different sizes of tower are used, depending upon the platform load;

(3) Travelling platform, which accommodates one or two wheel barrows depending on the size of the equipment. This is fitted with a safety device to minimize risk of damage and accident due to rope breakage.

The high speed 'Hoistex' which is most suitable and economical for construction of lofty buildings, is operated on a more or less permanent basis. One or more 'Hoistex' depending upon the size of the building, is carefully positioned and erected at the commencement of the work in order to avoid any shifting during the course of construction.

It ensures speedy and smooth handling of materials which is of prime importance on all constructional works. Large plant of this type is generally operated by a power driven winch but small hand operated unit may be used where load to be handled and the lift are not excessive.

Hoisting of building materials to storeys higher than the 4th floor by manual labour is not only very costly, but it involves great risk to labour and takes comparatively much longer time, which is detrimental to speedy execution of any job of magnitude.

Output

I have dealt with the efficiency of the various plants to some extent in the foregoing paras. Now I shall deal with their output. The output of a plant depends upon many factors viz ; (1) experience of the operator employed ; (2) fuel and lubricant used ; (3) maintenance, periodical repairs and renewals ; (4) nature of the work on which the plant is used i.e., correct application of plant ; and lastly, (5) the correct combination and matching of plant when more than one unit of plant are involved.

In order to get the maximum output, the plant should be worked and run efficiently by a competent operator using right type of fuel and lubricant and maintained properly upon which the life of a plant mainly depends. Periodical repairs and renewal of parts are to be done in time so that the plant does not sit idle for any unnecessary length of time. Care should also be taken to select the most suitable type and size of plant, giving due consideration to the nature of the work, the output of the plant selected per hour, the amount of work involved and the time in which it has to be carried out to ensure economical use of the plant. It would be uneconomical if a concrete mixer capable of producing 100 cft. of concrete per hour is used in a work where only 50 cft. per hour is required.

The unit cost of a work depends upon the actual output of the plant during the period it is used and the average working cost of plant per hour during that period. Therefore, the maximum the output and/or the minimum the working cost, the minimum is the unit cost of work i.e. the latter varies inversely with the output but directly with the working cost of the plant. Again, the working cost of the plant depends mainly upon the following factors, viz :—

(1) the capital cost of the plant ;

(2) the life of the plant ;

(3) the cost of repairs and renewals;

(4) the cost of fuel and lubricant consumed ; and

(5) the wages of the operator and helpers if any.

The working cost will be unnecessarily higher if the plant is over-worked or its maintenance is neglected involving higher cost of repairs and renewals and shortening the life of the plant itself. It will also increase if the plant is not required to work for the full working hours in each and every day it is used as the wages of the operator and the helper will have to be paid in full whether they work for the full or part time in a day.

It is, therefore, of vital importance for all engineers and builders to exercise utmost discretion and care in making purchase of a plant and in its operation so as to get the maximum output at the minimum working cost possible as the low unit cost of the work is not only reflected in the profit obtained but makes possible the submission of competitive tenders for future work. The output of the plant per hour also influences the time taken to carry out the work and so it will assist the builders in securing contracts in open competition, time being now-a-days the essential factor of contracts for all urgent and important constructions.

I. Comparison of Cost of Mixing Concrete in Mixing Plant and by Hand Mixing

A. Cost of Concrete in Mixing Plant—

Date :—Make : Winget, capacity : 7/10 cft.

A1.—Working cost of mixing plant :—

(*a*) Running cost per day of 8 working hours :—

	Rs.	a.	p.
(*i*) Consumption of Diesel oil—1 gallon	1	7	0
(*ii*) Consumption of lubricating oil			
(Change every fortnight)	Rs.		
1¼ Gallon @ Rs. 5 gallon ..	6	4	0
consumption per day ..	6	4	0
	0	7	3
	14		
(*iii*) Wages of Mixer Driver per day	4	0	0
	5	14	3

(*b*) Depreciation per day of 8 working hours :—

Data :—Cost Rs. .. 10,000
Deduct—scrap value .. 1,000
9,000

Life :—7 years.
Assuming 26 working days per month—

Depreciation per day of 8 hours Rs. $\frac{9,000}{7\times12\times26}$ = 4 2 0

(*c*) Cost of maintenance and repairs :—

	Rs.
1st 3 years @ Rs. 15 per month	540
Balance 4 years @ Rs. 40	1,920
Wages of attending mechanic (1/10th mechanic @ Rs. 200 per month for 7 years) =	1,680
	4,140

Cost of maintenance per day $= \frac{4,140}{7\times12\times26}$ = 1 14 6
11 14 9
Say 12 0 0

Working cost per day .. 12 0 0
Add 25% for the idle period .. 3 0 0
15 0 0

Output—

This varies according to the nature of work viz :—

(1) In complicated R.C. Columns 600 cft. day
(2) In R.C. floors and beams 800 ,,
(3) In R.C. foundation rafts 1,000 ,,

Average output per day of 8 hrs = 800 cft.

Unit working cost of plant = Rs. $\frac{12}{8}$ Rs. a. p. 1 8 0 cft.

A2. Mixing cost of concrete in a Concrete Mixer :—

	Rs.	a.	p.
(*i*) Cost of forms :—			
(3 Forms Rs. 9-8-0 each	28	8	0
Life 90 working days.)			
Cost per day of 8 hrs.	28 8 0 / 90 =	0	5	0
(*ii*) Cost of shovel—3 Nos @ 3/- =	9.		
Life—3 months—cost per day =	0	1	7
(*iii*) Watering cans and sundries, Say	0	0	4
(*iv*) Labour— 9 Mazdoors @ 2-8-0 =	22	8	0
Output being 800 cft. cost of labour % cft. =	2	13	0
	3	3	11
Say	3	4	0
	% cft.		

Unit cost of work (mixing concrete in mixing plant) 4 12 0 % cft. of concrete

B. Unit Cost of Work by Hand Mixing

	Rs.	Rs.	a.	p.
(*a*) Cost of forms—4 Nos. @ Rs. 9-8-0 =	38			
Life 3 months—cost per day =	38 / 90	0	7	0
(*b*) Cost of shovels—4 Nos. @ Rs 3 =	12			
Life 3 months—cost per day =	Rs. 12 / 90	= 0	2	2
(*c*) Cost of spades—4 Nos. @ Rs. 3 =	12			
Life 6 months—cost per day =	Rs. 12 / 180	= 0	1	0
(*b*) Watering cans and sundries, say		0	0	4
		0	10	6

Output being 750 cft., cost per % cft. — 0 10 6 / 7.5 = 0 1 6

Cost labour :—

	Rs. a. p.
A Mate @ Rs. 3 per diem	= 3 0 0
15 Mazdoors in Tagars. 1 Mazdoor for watering. 16 Nos. @ Rs. 2-8-0 per diem	=40 0 0
	43 0 0

Cost of labour per % cft. of concrete. — Rs. 43 / 7.5 = 5 11 6

Unit cost of work (mixing concrete by hand). — Rs. a. p. 5 13 0 % cft.

Conclusion :
Cost of mixing concrete in Mixing Plant is cheaper than that of hand mixing.

II. Comparison of Unit Cost of Vibrator and Hand Poking

A. Unit cost of Vibrator—Make : Wacker.

	Rs. a. p.
(1) Running cost per day of 8 hrs.	
(*a*) Consumption of petrol—1 gallon.	2 10 0
(*b*) Consumption of lubricating oil 1/32 gallons Say	0 2 0
(*c*) Wages of Driver—1 No. @ Rs. 3-8-0 day Wages of Helper—1 No. @ Rs. 2-8-0 day	5 8 0
	8 5 0

(2) Depreciation per day of 8 hrs :—
Cost of plant—Rs. 3,000.
Life 4 years of 300 working days/annum

Depreciation/day = Rs. 3,000 / 4×300 = Rs. a. p. 2 8 0

(3) Cost of maintenance and repairs :—
Renewals and repairs—10% of depreciation = 0 4 0

Wages of attending mechanic—say one day every month i.e.

Rs. a. p. 7 8 0 / 30 = Rs. a. p. 0 4 0 — 0 8 0

11 5 0

Output being 800 cft. per day

Unit cost of poking with Vibrator = 11 5 0 / 8 = 1 7 0 % cft. (say

B. Cost of poking without Vibrator (i.e. by manual labour).

6 skilled Mazdoors	@ 2 8 0 each	=15 0 0
1/4 Mason	@ 4 0 0	= 1 0 0
		16 0 0

(Average output being 800 Cft/day)

Unit cost of poking by manual labour — Rs. 16 / 8 = Rs. 2 % Cft.

III. Comparison of Cost of Hoisting Concrete by Mechanical Hoist and by Head Load

(As per observation made in respect of construction of New Secretariat Building in Calcutta—One Year's job).

A Unit cost of lift by mechanical hoist—

	Rs.
(1) Cost of hoist including mast =	18,000
Cost of extra rope	200
Cost of overhauling on completion of job	200
	18,400

	Rs.	Rs.
Less sale proceeds assuming the life of the hoist to be 6 years	15,000	
Amount of depreciation ..	3,400	3,400
(2) Cost of erection (labour) and sundries :—		
(*a*) Angle iron brackets—12 sets @	15 each	180
(*b*) Fixing and dismantling 12 set @	2 8 0 each	30
(*c*) Carriage of hoist components from and to Storeyard. 6 trips	@ 15 each	90
(*d*) Erection and dismantling of the hoist	1 set	160
(*e*) Making masonry pits for the machine and buckets. 2 nos. @	150 each.	300
(*f*) Making platform at different levels. 7 Nos.	@ 10 each	70
(*g*) Cost of Wheel barrows. 12 Nos.	@ 25 each	300
		1,130
Total cost for the whole job		4,530

Amount of work done—2/3rd of the total volume of concreting done.

5th to 13th floors (inclusive)—		cft.	cft.
—9 floors	@	18,210	1,63,890
14th and 15th floors—Nos.	@	9,100	18,200
			1,82090
2/3d of		1,82,090	121,393.
∴ Cost per % cft.		..	3 12 0 (say)

(3) Running cost per day of 8 hrs :—
Data—Average Quantity hoisted 800 cft./day.
Average lift 96 ft.

	Rs.	a.	p.
(*a*) Diesel oil 1½ gallons	2	7	0
(*b*) Lubricating oil. 1/16th gallon	0	5	6
(*c*) Grease ..	0	5	6
(*d*) Jute ..	0	4	0
(*e*) Petty repairs and sundries ..	1	0	0
(*f*) Wages :—			
One Driver	4	0	0
One attendant	2	0	0
Two wheel barrow pullers ..	10	0	0
	20	6	0

∴ Running cost per % cft. $= \frac{20\ 6\ 0}{8}$ = Rs. 2 9 0 (say)

∴ Unit cost of lift by mechanical hoist Over an average lift of 96 ft =(Rs. 3-12-0 plus Rs. 2-9-0=Rs. 6-5-0% cft

B. Lift by head load—
(By actual observation)

18 Nos Mazdoors lifted 500 cft. of concrete from ground to 2nd floor in a day.
No. of Mazdoors required % cft.=3.6 heads.

18 Nos. Mazdoors lifted 400 cft. of concrete from ground to 3rd floor in a day.

∴ No. of Mazdoor required % Cft.=4.5
∴ Extra labour required for 1 lift 0.9 heads.

	Rs.	a.	p.
@ Rs. 2 per day	1	13	0

Unit cost of lift by head load over an average lift of 95 ft. =8×Rs. 1-13-0
14-8-0% cft.

Conclusion—
Work by head load is uneconomical after 4th floor.

IV. Working Cost of Granulators

1. Running cost per day of 8 hrs :—

	Rs.	a.	p
(*a*) Consumption of Diesel Oil. 6 galls. @ Rs. 1-10-0 gall	= 9	12	0
(*b*) Lubricating oil. ¼ Gall. @ Rs. 6/-gall.	1	8	0
(*c*) Wages of a driver	4	0	0
(*b*) Salary of a Supervisor	6	0	0
	21	4	0

2. Depreciation per day :—

Cost	Rs. 30,000.
Life	8 years.
Idle period	4 months in a year.
Working days	25 days per month.
Scrap value	1,500.

Depreciation per day=Rs. 30,000.

$\frac{\text{Rs. } 1{,}500}{8\times8\times25}$ = Rs. a p. 17 13 0

3. Cost of maintenance and repairs :—
(including wages of attending mechanic).
1st 3 years@ 2% on capital cost

$= \frac{2}{100} \times 30{,}000 \times 3$ = Rs. 1,800.

4th to 10 th years @ 7½ % on capital cost

$= \frac{7.5}{1.00} \times 30{,}000 \times 7$ = Rs. 15,750

17,550

Cost per day of 8 hrs. $= \frac{17{,}550}{8\times8\times25}$ = 11 0 0

50 1 0

4. Loading and charging chelis into crushers, collecting and stacking.
10 Mazdoors @ Rs. 2 = 20 0 0

Total Working cost = 70 1 0

Output per day of 8 hrs.—
@ 9 tons per hr.=9×20×8=1,440 cft.
Unit cost of breaking to 3/4″ size chips in a Granulator

$\frac{70\ 1\ 0}{14.4}$ Say 5 0 0 % cft.

Against the usual rate of Rs. 18 to Rs. 20 % cft. by manual labour.

Conclusion—
Crushing by Granulators is very economical.

From Symposium on "Housing and Building Materials" Vol. 2. published by National Buildings Organisation.

Continued from page 11

Design Criteria for Power Tunnels in Rock

(2) Provision of flap valves in holes drilled through the lining. These valves are designed to open only towards inside of the tunnel and thus relieve external pressure when necessary. As already mentioned, in case of high internal pressure in tunnels excavated in poor type of rocks, a steel liner is invariably provided to eliminate leakage and make the tunnel water tight.

Where conditions permit, main supply tunnel may be constructed with slight grade in the upper reaches and a steep short steel lined tunnel provided at the end for connecting it to surge tank or power house. This will reduce the hydrostatic pressures inside the tunnel to an appreciable extent, resulting in reduction of external pressures that are built up due to seepage through surrounding material. The provision of this steep portion may cause constructional difficulties, but may ultimately be cheaper as steel lining of this short portion will be far less costly than placing liner in the entire length.

Mandira Dam for Rourkela Plant

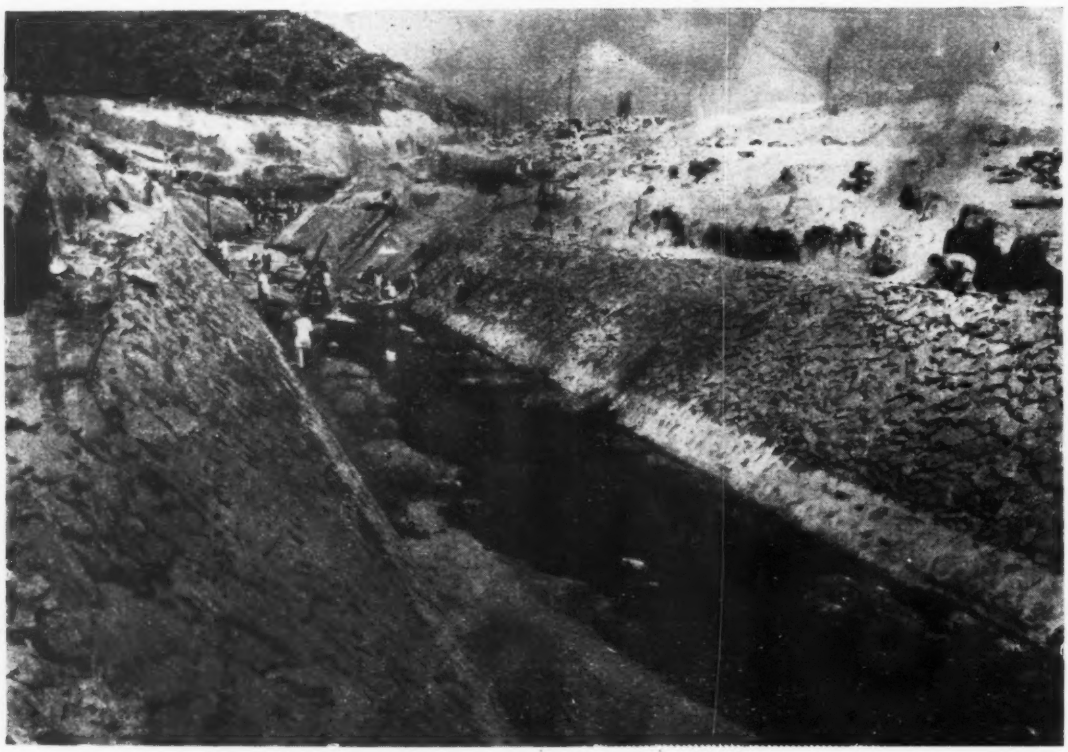
Across the river Sankh at Mandira, in Orissa, the construction of a dam is making steady progress. The dam will ensure perennial supply of water to the Rourkela steel plant.

The original idea suggested by the Orissa Government was to put up a masonary dam at Mandira. But since the foundations were unfavourable for a masonary dam, the Central Water & Power Commission and the Hirakud authorities explored the possibility of an earthen dam. It is this one that is now coming up.

The dam site is 16 miles upstream from the Rourkella plant. It will be 1365 feet long and 90 feet high and is estimated to cost Rs. 1.9 crores.

The work, which has been entrusted to the Chief Engineer, Hirakud Project, is going according to schedule and is expected to be completed this

Work at Mandira Dam in progress



[*Continued on page* 25

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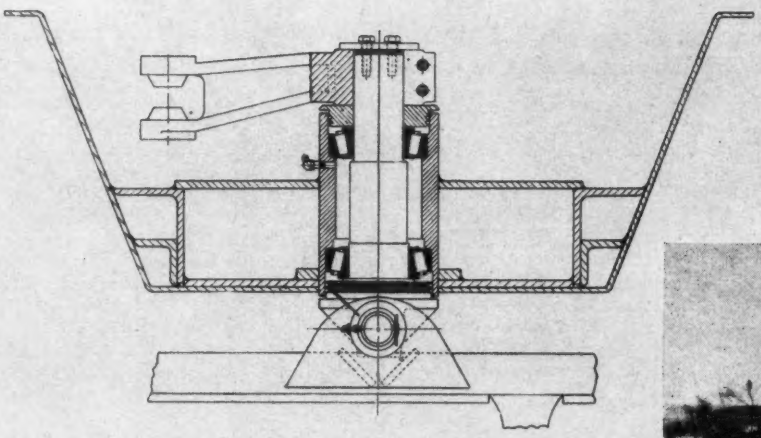

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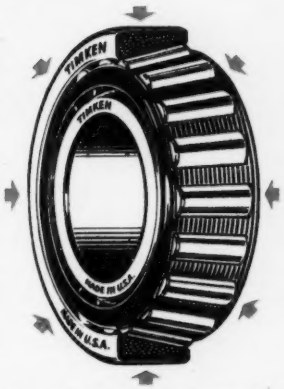

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Calculation with the Momentum Theorem

The application of the momentum theorem and its modified form of the total-force equation is explained for hydraulic calculations with particular regard to the conditions in channels of rectangular cross section. Further the frequently adopted simplification, the advantage in calculating with relative numbers for this case and its relation to the energy principle are considered. The methods are applicable to other cross sections also.

By Dr. Ing. P. G. FRANKE

1. The fundamental equations

The momentum theorem derived by I. Newton can also be applied for the steady flow, expressing that the timely change of momentum on a limited zone of flow is equal to the sum of the external forces concerned.

In the case of a straight flow all terms can be related to the direction of flow instead of writing in vectors. Then the general form is given by

$$K = \frac{d}{dt} \Sigma\ (mv) = \frac{dJ}{dt} \qquad (1)$$

Denoting the cross sections under consideration in the direction of flow with (I) and (II) the relation

$$\frac{dJ}{dt} = \rho\, Q\ \bar{\alpha}\,(v_{II} - v_I) = K \qquad (2)$$

is obtained, in which the terms are as under :

J = momentum of the mass
ρ = density of the fluid
Q = discharge in unit of time
$\bar{\alpha}$ = correction factor
v = mean velocity in the cross sectional area
K = sum of external forces.

The factor $\bar{\alpha}$ is measure of the distribution of momentum across the section concerned. It is analogous to the correction factor α for the mean velocity head. Between these two factors exists the relationship

$$\alpha = 1 + 3\eta = 3\,\bar{\alpha} - 2$$

or (3)

$$\bar{\alpha} = 1 + \eta = \tfrac{1}{3}(\alpha - 2).$$

Generally it is $1.01 \leq \bar{\alpha} \leq 1.05$ corresponding to the values of $1.03 \leq \alpha \leq 1.15$.

In the sections (I) and (II) may be straight stream threads so that a correction factor regarding the pressure effect of curvature is not to be introduced.

2. The total-force equation

It may be denoted with P^* the sum of all external forces which are acting on the mass between the two sections of reference except the hydrostatic pressure forces on these areas. Then equation (2) can be expressed as

$$\rho\, Q\ \bar{\alpha}\,(v_{II} - v_I) = p_I F^I - p_{II} F_{II} + P^* \qquad (4)$$

or modified into

$$\gamma\left(\frac{\bar{\alpha}}{g} Q v_{II} + \frac{p_{II}}{\gamma} F_{II}\right) - \gamma\left(\frac{\bar{\alpha}}{g} Q v_I + \frac{p_I}{\gamma} F_I\right) = P^* \qquad (5)$$

where with p are denoted the pressures and with the cross sectional areas. The expression

$$\gamma\left(\frac{\bar{\alpha}}{g} Q v + \frac{p}{v} F\right) = \gamma S^* \qquad (6)$$

was introduced by A. Koch as support force or abutment force (Stützkraft). Subsequently the equation (5) can be expressed in the short form

$$\gamma\,(S_{II}^* - S_I^*) = P^* .$$

With the equation (5) or (7) the momentum theorem for steady flow is only rewritten in another form, the so-called total-force equation

3. The conditions for $P^* = O$

In the case of $P^* = O$, which may be real or assumed, it follows $S_{II}^* = S_I^*$ or $S^* = \text{const}$ for the zone concerned. From the relation

$$S^* = \frac{\bar{\alpha}}{g} Q v + \frac{p}{\gamma} F = \frac{\bar{\alpha}}{g} \frac{Q^2}{F} + \frac{p}{\gamma} F$$

the equation for the discharge

$$Q=F\ v=\sqrt{\frac{g}{\alpha}F\left(S^*-\frac{p}{\gamma}F\right)} \qquad (8)$$

$$=F\sqrt{\frac{g}{\alpha}\left(\frac{S^*}{F}-\frac{P}{\gamma}\right)}$$

is obtained. Further considerations will be done for the particular case of a rectangular channel-profile only.

4. Application to rectangular channels

4.1 *The total-force equation*

For flumes it is convenient to consider all values for the unit of width. Then the discharge per meter width is given as

$$q=\frac{Q}{b}=y\ v\ .$$

With the further terms

$$y=\frac{F}{b}\ ;\ \frac{p}{\gamma}=\frac{y}{2}\ ;\ \frac{P^*}{b}=P \text{ and } \frac{S^*}{b}=S$$

the equations (5) and (6) will be modified into

$$\gamma\left(\frac{\alpha}{g}\,q\,v_{II}+\frac{y_{II}^2}{2}\right)-\gamma\left(\frac{\alpha}{g}\,q\,v_I+\frac{y_I^2}{2}\right)=P \qquad (9)$$

and

$$\gamma\left(\frac{\alpha}{g}\,q\,v+\frac{y^2}{2}\right)=\gamma\,S\ . \qquad (10)$$

Subsequently for equation (9) the short relation

$$\gamma\,(S_{II}-S_I)=P \qquad (11)$$

will be obtained.

This consideration can also be applied with fair approximation for other very large channels ($b>>y$) for which the value y represents the average depth.

4.2. *The conditions for $P=0$*

4.2.1. *The general equations*

For this simplified case is following $S_{II}=S_I$ or $S=$const between the two reference sections. From the relation

$$S=\frac{\alpha}{g}\,q\,v+\frac{y^2}{2}$$

or

$$S=\frac{\alpha}{g}\,v^2y+\frac{y^2}{2}=\frac{\alpha}{g}\,\frac{q^2}{y}+\frac{y^2}{2} \qquad (12)$$

follows for the discharge per unit of width the expression

$$q=y\,v=\sqrt{\frac{g}{\alpha}\,y\left(S-\frac{y^2}{2}\right)}$$

$$=y\sqrt{\frac{g}{\alpha}\left(\frac{S}{y}-\frac{y}{2}\right)} \qquad (13)$$

Form this equation it is obvious that for each possible discharge, with exception for its maximum amount there are corresponding two positive values for depth.

In the particular cases when $y=0$ and $y=\sqrt{2S}$ it will be $q=0$. Between these two limits an extreme value lies, which is a maximum. Critical depth y_c and critical velocity v_c are corresponding to q_{max}. Over this stage is the subcritical region ($y'>y_c$ and $v'<v_c$) and below it is the supercritical region ($y''<y_c$ and $v''>v_c$).

Regarding that the velocity head will be expressed as $k=\frac{v^2}{2g}$ equation (12) can be modified accordingly and the relation

$$S=2\alpha ky+\frac{y^2}{2}=y\,(2\alpha k+\frac{y}{2}) \qquad (12a)$$

will be obtained.

4.2.2. *The critical depth y_c for $S=$const*

From equation (13) follows for $\frac{dq}{dy}=0$ the relation

$$y=\sqrt{\tfrac{2}{3}S}=0.8165\sqrt{S}=y_c \qquad (14)$$

The critical depth y_c is the specific depth for the maximum discharge which is possible under the conditions represented by S in the section concerned.

4.2.3. *The maximum discharge q_{max} for $S=$const*

Continuing from equation (13) for the maximum discharge

$$q_{max}=\sqrt{\frac{g}{\alpha}y_c\left(S-\frac{y_c^2}{2}\right)}=y_c\sqrt{\frac{g}{\alpha}\left(\frac{S}{y_c}-\frac{y_c}{2}\right)}$$

two expressions can be derived, as under :

Substituting $y_c=\sqrt{\frac{2}{3}S}$ into the above formula the expression

$$q_{max}=\sqrt{\frac{g}{\alpha}}\sqrt[4]{(\tfrac{2}{3}S)^3}=\frac{2.311}{\sqrt{\alpha}}\sqrt[4]{S^3} \qquad (15)$$

and with $S=\frac{3}{2}y_c^2$ the relation

$$q_{max}=y_c\sqrt{\frac{g}{\alpha}\,y_c}=\frac{3.132}{\sqrt{\alpha}}\,y_c\sqrt{y_c} \qquad (16)$$

will be obtained.

4.2.4. *The critical velocity* v_c

Regarding the general relation for the maximum discharge

$$q_{max}=y_c\ v_c$$

for the critical velocity the expressions

$$v_c=\sqrt{\frac{g}{\alpha}}\ \sqrt[4]{\tfrac{2}{3}S}=\frac{2.830}{\sqrt{\alpha}}\ \sqrt[4]{S} \qquad (17)$$

and

$$v_c=\sqrt{\frac{g}{\alpha}y_c}=\frac{3.132}{\sqrt{\alpha}}\ \sqrt{y_c} \qquad (18)$$

will be directly obtained from the equations (15) and (16).

4.2.5. *The minimum value* S_{min} *for* $q=const$

The minimum conditions for a certain discharge are only accomplished if $q=q_{max}$ and $y=y_c$.

From equation (16) it is following

$$q^2_{max}=\frac{g}{\alpha}\ y_c^3=q^2$$

hence

$$y_c=\sqrt[3]{\frac{\alpha}{g}\ q^2}=\sqrt{\tfrac{2}{3}\ S_{min}} \qquad (19)$$

and further

$$S_{min}=\tfrac{3}{2}\ \sqrt[3]{\left(\frac{\alpha}{g}\ q^2\right)^2}=\tfrac{3}{2}\ y_c^2 \qquad (20)$$

This derivation will be also obtained referring equation (12) for the minimum at $\frac{dS}{dy}=0$.

4.3. *The S-line*

Considering a constant discharge the value S will vary according to the depth only. When the values of S are plotted against those of the depth y the line $S=S(y)$ is obtained.

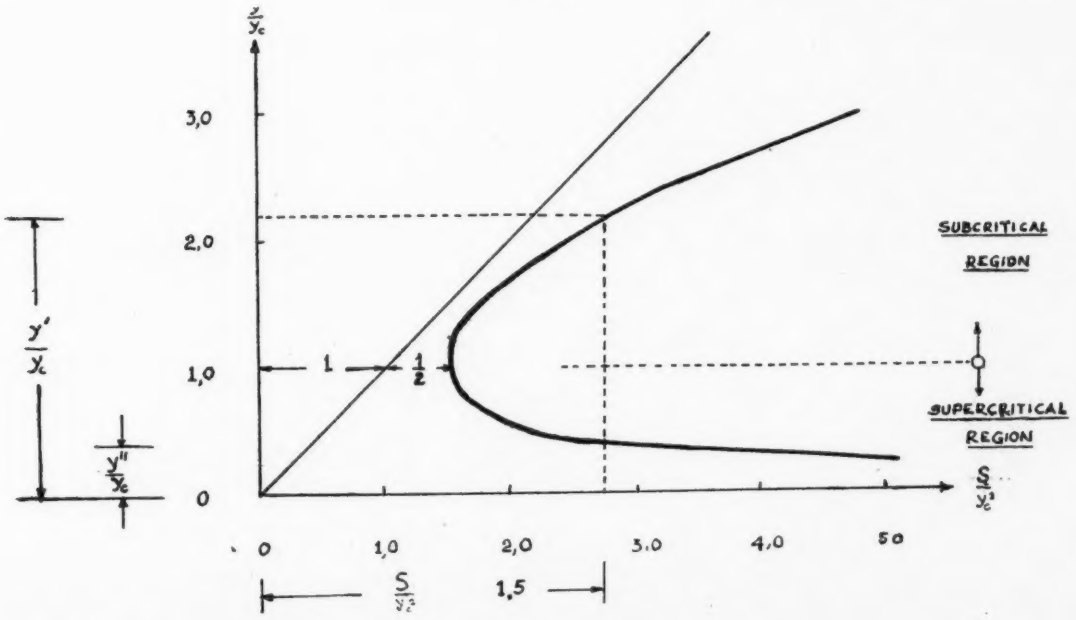
For the case of a dimensionless form the equation (12) can be divided by the square of the critical depth y_c^2. With the substitution of $q^2=\frac{g}{\alpha}\ y_c^3$ which is derivated from equation (19) follows the relation

$$\frac{S}{y_c^2}=\frac{y_c}{y}+\tfrac{1}{2}\frac{y^2}{y_c^2} \qquad (21)$$

For each discharge the critical depth can be calculated and subsequently the conditions for a desired depth y, either in the subcritical or in the supercritical region, will be found out.

Some relative numbers are contained in table 1 and the diagram is shown in figure 1.

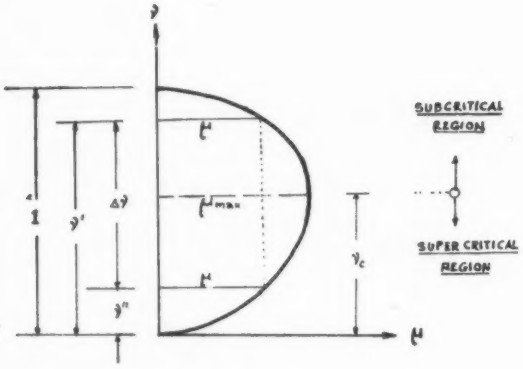
Figure 1



4.4. *The q-line*

If the values of q for S=const are drawn in relation to the water depth y the diagram of the discharge-line is obtained (fig. 2).

Figure 2



It is obvious that for all discharges $q<q_{max}$ two alternative depths are corresponding, one in the subcritical region and the other in the supercritical region. Hence follows for S=const and $q<q_{max}$ the relation

$$S=\frac{\alpha}{g}\,q\,v'+\frac{y'^2}{2}=\frac{\alpha}{g}\,q\,v''+\frac{y''^2}{2}$$

or (22)

$$\frac{\alpha}{g}\,q\,(v''-v')=\tfrac{1}{2}\,(y'^2-y''^2)$$

for which $\alpha'=\alpha''=\alpha$ is applied. With the equation of continuity $q=v'\,y'=v''\,y''$ and the simplification $\alpha=1$ the expressions

$$y'^2+y'\,y''-\frac{2}{g}\,v''^2\,y''=0$$

or (23)

$$y''^2+y''\,y'-\frac{2}{g}\,v'^2\,y'=0$$

will be obtained. Subsequently the corresponding water depth can be expressed as

$$y'=-\frac{y''}{2}(\pm)\sqrt{\frac{y''^2}{4}+\frac{2v''^2y''}{g}}=-\frac{y''}{2}+\sqrt{\frac{y''^2}{4}+4y''k''}$$

and (24)

$$y''=-\frac{y}{2}(\pm)\quad\frac{y'^2}{4}+\frac{2v'^2y'}{g}=-\frac{y'}{2}+\sqrt{\frac{y'^2}{4}+4y'k'}$$

where $k=\frac{v^2}{2g}$ is the velocity head concerned. With the term $M=\frac{v}{\sqrt{gy}}$ or $M^2=2\,\frac{k}{y}$ respectively, these equations can be modified accordingly.

From the equations (24) the difference of water depths.

$$\triangle y=y'-y''$$

and the relative values

$$\frac{y'}{y''}\quad\text{or}\quad\frac{y''}{y'}$$

and

$$\frac{\triangle y}{y'}=1-\frac{y''}{y'}\quad\text{or}\quad\frac{\triangle y}{y''}=\frac{y'}{y''}-1$$

can be derived easily.

4.5. *The proportional factors* ν and μ

Rewriting equation (13) in the following form

$$q=\frac{\sqrt{g}}{\sqrt{\alpha}}\;y\;\frac{\sqrt{2S}}{\sqrt{2S}}\;S\left(1-\frac{y^2}{2S}\right)$$

and substituting

$$\nu=\frac{y}{\sqrt{2S}} \tag{25}$$

the dimensionless form

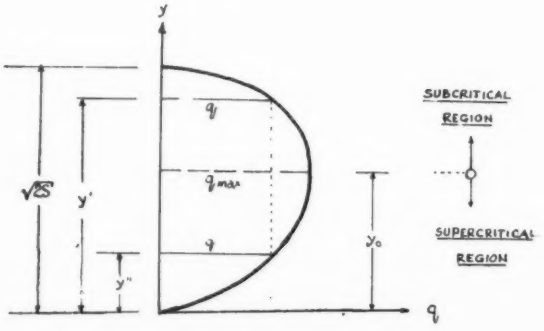
$$\frac{q}{\sqrt{\frac{g}{\alpha}}\sqrt[4]{2S}\,\sqrt{S}}=\sqrt{\nu}\,\sqrt{1-\nu^2}=\sqrt{\nu-\nu^3}=\mu \tag{26}$$

is obtained, in which the limits are

$$\underset{=}{0<\nu<1}\qquad\text{and}\qquad\underset{=}{0<\mu<0{,}620403.}$$

For the numbers ν the corresponding values μ can be worked out once for all (table 2). Instead of each particular q-line, related to every value S, only a single unit-q-line or μ-line is obtained (figure 3)

Figure 3



The critical value between mild and steep flow is

$$\nu_c = \tfrac{1}{3}\sqrt{3} \qquad (27)$$

and for the expected depth of water follows

$$y = \nu \sqrt[4]{2S}$$

as a general relation.

Referring to equation (26), the discharge is given by the expression

$$q = \mu \sqrt{\frac{g}{\alpha}} \sqrt[4]{2S^3} \qquad (28)$$

with $\mu_{max} \sqrt{g}\ ^4/2 = 0.6204 \times 3.1320 \times 1.1892 = 2.311$ for the maximum discharge

$$q_{max} = \frac{2.311}{\sqrt{\alpha}} \sqrt[4]{S^3}$$

is obtained again.

The difference of water depth between the two corresponding values is expressed as

$$\Delta y = (\nu' - \nu'')\sqrt{2S} = \Delta\nu \sqrt{2S} \qquad (29)$$

The numbers $\Delta\nu$ can also be worked out directly considering that for each value $\mu < \mu_{max}$ exists the condition

$$\mu = \sqrt{\nu' - \nu'^3} = \sqrt{\nu'' - \nu''^3}$$

With these proportional factors the following relations

$$\frac{y'}{y''} = \frac{\nu'}{\nu''} \quad \text{or} \quad \frac{y''}{y'} = \frac{\nu''}{\nu'}$$

and

$$\frac{\Delta y}{y'} = 1 - \frac{\nu''}{\nu'} \quad \text{or} \quad \frac{\Delta y}{y''} = \frac{\nu'}{\nu''} - 1$$

will also be obtained.

4.6. *The proportional factors n and m*

From the general equation

$$S = \alpha \frac{q}{g} v + \frac{y^2}{2} = \alpha_c \frac{q_{max}}{g} v_c + \frac{y_c^2}{2}$$

it follows with $\alpha_c = \alpha$ the relation

$$\frac{\alpha}{g} \frac{q^2}{y} + \frac{y^2}{2} = \frac{\alpha}{g} \frac{q^2_{max}}{yc} + \frac{yc^2}{2}$$

Multiplying with $\frac{y}{yc^3}$ and regarding that $yc^3 = \frac{\alpha}{g} q^2_{max}$ the expression

$$\frac{q^2}{q^2_{max}} + \tfrac{1}{2} \frac{y^3}{yc^3} = \tfrac{3}{2} \frac{y}{yc}$$

is derived. Substituting the relative values

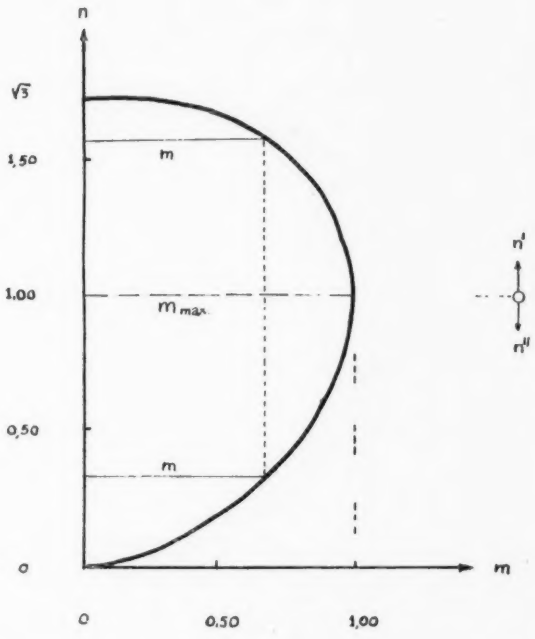
$$n = \frac{y}{yc} \quad \text{and} \quad m = \frac{p}{q_{max}} \qquad (30)$$

in the above the relation

$$m = \frac{n}{2}(3 - n^2) \qquad (31)$$

will be obtained. For all numbers $0 \leq n \leq \sqrt{3}$ the corresponding values $0 \leq m \leq 1$ can be worked out (table 3). The graph (figure 4) representing the curve $m = m(n)$ is similar to that of the μ-line.

Figure 4



If the value m is known, the depth concerned will be found out from the relations

$$y' = n'yc \quad \text{or} \quad y'' = n''yc$$

and for each value of n the factor m and the discharge

$$q = m\ q_{max}$$

can be obtained.

Further to each relative number, except $n = 1$, pertains a value $\Delta n = n' - n''$.

From the relation

$$m^2=\tfrac{3}{2}\,n'-\frac{n'^3}{2}=\tfrac{3}{2}\,n''-\frac{n''^3}{2}$$

the expressions

$$\triangle n=\tfrac{3}{2}\,n'\,(\pm)\;\sqrt{(\tfrac{3}{2}\,n')^2-3(n'^2-1)}$$

and (32)

$$\triangle n=-\tfrac{3}{2}\,n''\,(\pm)\;\sqrt{(\tfrac{3}{2}\,n'')^2-3(n''^2-1)}$$

are obtained.

4.7. *The relations between H and S*

The head of energy above the channel floors is expressed as

$$H=y+\frac{\alpha}{2g}\,v^2=y+\frac{\alpha}{2g}\,\frac{q^2}{y^2}$$

hence the discharge per unit of width is given as

$$q=y\,\sqrt{\frac{2g}{\alpha}\,(H-y)}\quad .$$

Further follows for the discharge from the simplified total-force equation

$$q=y\,\sqrt{\frac{g}{\alpha}\left(\frac{S}{y}-\frac{y}{2}\right)}\quad .$$

With $\alpha=\alpha'$ the relation between H and S can be expressed as

$$H=\tfrac{1}{2}\left(\frac{S}{y}+\tfrac{3}{2}\,y\right)$$

or (33)

$$S=y(2\,H-\tfrac{3}{4}\,y).$$

To the two corresponding depths y′ (subcritical region) and y″ (supercritical region) for a discharge $q<q_{max}$ is following

and	$S\pm$ const	for	H=const
	$H\pm$ const	for	S=const.

The q_h -line for H=const* and the q_s -line for S=const may be compared with the m-lines concerned (figure 5). Some reference values are contained in table 4.

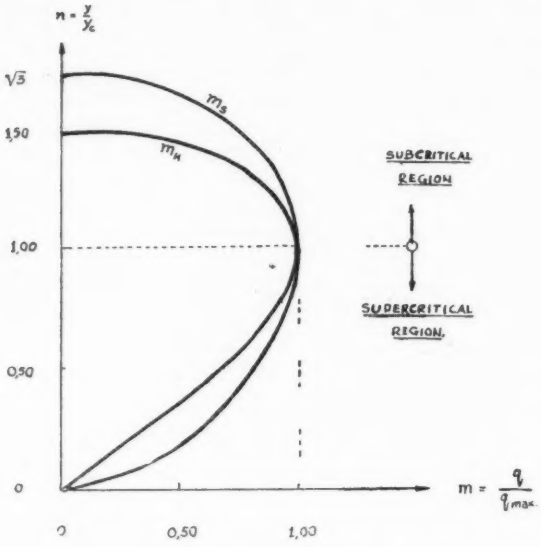


Figure 5

The calculations of the corresponding depths are based on the conditions concerned in *one* section. However, such considerations can be applied as an *approximation* for the relations between two sections where a change of the flowing state takes place. Denoting these two sections, both with parallel stream threads, in the direction of flow with (I) and (II) the following conditions will be obtained.

Accelerated motion :

$S_I>S_{II}$	for	H=const
$H_I<H_{II}$	for	S=const

Retarded motion :

$H_I>H_{II}$	for	S=const
$S_I<S_{II}$	for	H=const

The two lines representing the gradients of H and of S above the floor cannot ascend in the direction of flow. Therefore in a channel with horizontal floor it will be

$$\triangle H=H_I-H_{II}>0\quad\text{and}\quad\triangle S=S_I-S_{II}>0\ .$$

For approximations it can be assumed $\triangle H=0$ or $\triangle S=0$, however the other difference ($\triangle S$ or $\triangle H$) must be a positive value. From this consideration it is obvious that only the above first cases of each kind of motion are real ones.

*Franke, P. : Calculation with the Energy Head. Indian Journal of Power & River Valley Development VIII (1958) No. 2, pp. 19-24 and 26.

Considering for example the accelerated motion with change of the flowing state in the case of a discharge below a gate it can be assumed in the first instance $\triangle H=O$ and from the relation $\gamma(S_I-S_{II})=P$ the forces upstream can be found out. With sufficient exact empirical coefficients the loss of head in this zone will be regarded also.*

In the case of a retained motion with change of the flowing state occurs the hydraulic jump. The approximation with $\triangle S=O$ offers a possibility to find out the losses of head in this zone, which mainly occur due to the top roller. The other losses which can be throughout noteworthy, are not included, as the conditions for S=const are a simplification of the momentum theorem.

4.8. *Illustration*

In a flume the discharge per unit of width is given, as q=3.00,m³/sec m. For the considerations it may be assumed $\alpha=1$ and $\vec{\alpha}=1$. Then the cirtical depth for the minimum conditions ($H=H_{min}$ and $S=S_{min}$) will be $y_c=0.972=0.97$ m.

The corresponding water depths and the values concerned of H and S for

$$y=2.150=2.15 \text{ m}$$

and

$$y=0.515=0.52 \text{ m}$$

are to be found out, considering for each case once S=const and once H=const.

The given values

$q=\frac{Q}{b}$	m³/sec m	3.00	
$y \gtrless y_c$	m	2.15	0.52
$v=\frac{q}{y}$	m/sec	1.40	5.82
$k=\frac{v^2}{2g}$	m	0.10	1.73
$S=y\left(2k+\frac{y}{2}\right)$	m³/m	2.74	1.92
$H=y+k$	m	2.25	2.25

These two depths $y' > y_c$ and $y'' < y_c$ are the corresponding values for H=const. The difference in the depth concerned between mild and steep flow will be $\triangle y=y'-y''=1.63$ m.

The corresponding depth y″ for S=const

$y'=2.15$ m and $S=2.74_{12}$ m³/m
$H'=2.25$ m

Relative numbers

$$\nu'=\frac{y'}{\sqrt{2S}}=\sigma.9182$$

$$\mu=\sqrt{\nu'-\nu'^3}=\sqrt{\nu''-\nu''^3}=\sigma.3795$$

$$\nu''=\sigma.147\sigma$$

Water depth $y''=\nu''\sqrt{2S}=0.343=0.34$ m

Difference in depth $\triangle y=y'-y''=1.81$ m

Mean velocity $v''=\frac{q}{y''}=8.75$ m/sec

Mean velocity head $k''=0.051\ v''^2=3.91$ m

Energy head $H''=y''+k''=4.25$ m

The corresponding depth y′ for S=const

$y''=0.515=0.52$ m and $S=1.9200$ m³/m
$H''=2.25$ m
$\nu''=0.2628$; $\mu=0.4945$; $\nu'=0.8425$
$y'=1.65$ m $\triangle y=y'-y''=1.13$ m
$v'=1.82$ m/sec $k'=0.174=0.17$ m
$H'=1.82$ m

From the above numerical example the relations between H and S are obvious.

5. Conclusion

The momentum theorem or its transformed expression of the total-force equation respectively, is generally applicable for all conditions of flow. Naturally it is not always possible to consider all external forces exactly, this especially for the flow in open channels. The simplification with $P^*=O$ (or $P=O$ in flumes) is frequently a useful guide; however, it must be considered and noted as an approximation only.

* Franke, P. : Discharge below Vertical Gates in Flumes. Current Engineering Pratice I (1958) No. 1, pp. 39—42

Continued from page 18

Mandira Dam for Rourkela Plant

year. Supply of water will be ensured in the summer of 1959.

The construction of a regulator and the fall in the supply sluice have been practically completed and the river has been diverted through the regulator during the middle of February, 1958; 11,70,540 cu.ft. of earthwork on the main dam and 18,78,500 cu. ft. of earthwork on the spillway were done during this month.

The Rourkela plant which is expected to produce its first pigiron towards the end of 1958, is situated on the Brahmani. When the location was decided on, it was expected that the Brahmani could supply all the water necessary. But in 1955 and 1956, the minimum flow in the river was low and inadequate for the requirements of a steel plant.

This was perhaps exceptional but then the planners could not risk the possibility of a recurrence of scarcity of water.

It was necessary to ensure a minimum flow of 100 to 125 cusecs during the summer months. It was clear that a weir on the Brahmani would not do. It was, therefore, that the decision was taken to dam one of the tributaries of the Brahmani some way upstrean. The choice of the site fell on Mandira.

Employing a total of 939 skilled and 1991 unskilled workers, the dam when completed will ensure regular water supply to the first of India's three steel projects in the public sector.

Table 1 : *Values* $\frac{y}{y_c}$ *and* $\frac{S}{y_c^2}$

$\frac{y}{y_c}$	$\frac{S}{y_c^2}$
5.00	12.7000
4.00	8 2500
3.00	4.8333
2.90	4.5498
2.80	4.2771
2.70	4.0151
2.60	3.7643
2.50	3.5250
2.40	3.2967
2.30	3.0796
2.20	2.8745
2.10	2.6812
2.00	2.5000
1.90	2.3313
1.80	2.1755
1.70	2.0332
1.60	1.9050
1.50	1.7917
1.40	1.6943
1 30	1.6142
1.20	1.5533
1.10	1.5141
1.00	1.5000
0.90	1.5161
0 80	1.5700
0.70	1.6736
0.60	1.8467
0.50	2.1250
0.40	2.5800
0.30	3.3783
0.20	5.0200
0.10	10.0050
0	∞

Table 2 : *Values* V *and* μ

V	μ	V	μ
1.00	0	0.50	0.6124
0.95	0.3043	0.45	0.5991
0.90	0.4135	0.40	0.5797
0.85	0.4857	0.35	0.5542
0.80	0.5367	0.30	0.5225
0.75	0.5728	0.25	0.4841
0.70	0.5975	0.20	0.4342
0.65	0 6127	0.15	0.3829
0.60	0.6197	0.10	0.3146
$\frac{1}{2}\sqrt{3}$	0.6204	0.05	0.2233
0.55	0.6194	0	0

Table 3 : Values n and m

n	m	n	m	n	m
$\sqrt{3}$	0	1.15	0.9821	0.55	0.8613
1.70	0.3058	1.10	0.9922	0.50	0,8292
1.65	0.4785	1.05	0.9981	0.45	0.7934
1.60	0.5933	1.00	1.0000	0,40	0.7537
1.55	0.6805	0.95	0 9982	0.35	0.7096
1.50	0.7500	0.90	0.9927	0.30	0.6607
1.45	0.8067	0.85	0.9838	0.25	0.6060
1.40	0.8532	0.80	0.9716	0.20	0.5441
1.35	0.8915	0.75	0.9561	0.15	0.4726
1 30	0.9228	0.70	0.9373	0.10	0.3867
1.25	0.9479	0 65	0.9153	0.05	0.2738
1.20	0.9675	0.60	0.8899	0	0
1.15	0.9821	0.55	0.8613		

Table 4 : Values n′, n″ and $\triangle n$ related to $O \leqq m \leqq 1$ *for H=const and for S= const*

m	$H=const$			$S=const$		
	n'	n''	$\triangle n$	n'	n''	$\triangle n$
1.000	1.0000	1.0000	0	1.0000	1.0000	0
0.950	1·1708	0·8068	0.3640	1.2451	0.7329	0.5122
0.900	1.2341	0·7211	0.5130	1.3374	0.6191	0.7183
0,850	1.2793	0·6531	0.6262	1.4038	0.5318	0.8720
0,800	1·3149	0·5945	0·7204	1.4564	0.4589	0.9975
0·750	1.3444	0.5418	0.8026	1.5000	0.3956	1.1044
0,700	1,3693	0·4933	0.8760	1.5370	3.3397	1.1973
0,650	1.3908	0.4482	0·9426	1.5689	0.2898	1.2791
0.600	1·4094	0.4056	1.0038	1.5966	0.2449	1.3517
0.550	1.4256	0.3651	1.0605	1.6207	0.2045	1.4162
0.500	1·4397	0.3264	1·1133	1.6418	0.1683	1.4735
0·450	1.4520	0·2892	1·1628	1.6601	0.1358	1.5243
0·400	1.4626	0.2533	1.2093	1.6760	0.1071	1.5689
0.350	1·4717	0·2186	1.2531	1.6897	0.0818	1.6079
0·300	1·4794	0.1850	1.2944	1.7012	0.0601	1.6411
0·250	1·4858	0·1523	1.3335	1.7108	0.0417	1.6691
0·200	1.4910	0·1204	1.3706	1.7186	0.0267	1.6919
0·150	1·4950	0·0893	1.4057	1.7245	0.0150	1.7095
0·100	1.4978	0.0589	1.4389	1.7287	0.0067	1.7220
0.050	1.4994	0.0292	1.4702	1.7312	0.0017	1.7295
0	1.5000	0	1.5000	1.7321	0	1.7321

Dams across Valleys in Central Germany

By GERHARD H. KEGEL

The area between Harz, Elbe and Saale takes an important rank in the national economy of the German Democratic Republic. Fertile soils and rich deposits of brown coal, ores and salts are to be found here. Through the constant upswing of industry and agriculture one problem which had already been existing for decades was, however, aggravated. By nature, all water resources, rain and discharge of water are unevenly distributed in the Central German area. In the headwaters of the rivers the water reservoirs have been decreasing in quantity and quality for decades. The subsoil water has been constantly decreasing, and during the periods of growth the plants can no longer draw sufficient moisture from the soil. And there is good reason for this.

The Harz Mountains which is 900 meter high on the average, is rising from the plains quite abruptly as a wood of 20 kilometers length, with its peak, the Brocken, which is even 1,142 meters high. These woody mountains hinder the way of the rain clouds drifted from the Atlantic Ocean, so that they cannot enter the fertile eastern foreland. The function of the Harz as a weather frontier is such a thorough one that the average yearly rain in the foreland of the Harz and in the plain land does not even come up to 500 millimeters, whereas almost threefold the rain quantity is discharged over the western part of the mountains. This happens often with such an elementary force that the Harz rivers are rising and become torrents of high water.

Until very recently the Harz towns had to record many catastrophes caused by high water. Water became the life and death of the population. The high water used to flow off rapidly, causing a lot of damage, and at the end of a rain period there was very often a draught.

The water level in the Harz rivers used to fall so rapidly that within a few days the torrents of foaming rivers changed into narrow streamlets which were neither big enough for the irrigation of fields and meadows, nor for the propulsion of water-power plants. The rivers could not take up any waste waters and could not serve any industrial purposes.

Already after the high water catastrophe of 1897 the population hoped to be freed from this constant danger by the construction of dams across the valleys. But the princes of Brunswick, Blankenburg and Stollberg who owned the land, did not think much of building such dams across the valleys. They did not see any profits for themselves resulting from these projects, and they did not care for the wretched situation of the population.

The question of the utility of water regulation also frustrated many other projects in later years. Even the foundation of a hydrological association formed by all communities and towns threatened by high water, was not of much avail. Those interested in the utilisation of the water, got into a quarrel. For according to their point of view, the resolution to place money at the disposal for building dams across the valleys was solely dependent on the profitability of water storage management and of the utilisation of the stored quantities of water. Thus each competitor pursued his own selfish aims of profit without considering the interest of the population in the construction of such dams.

For decades the population of the Harz and moreover the inhabitants of a large area of Central Germany had to wait in vain. After the formation of the German Democratic Republic the great projects of water regulation in the Central German area were put on the agenda already under the first Five Year Plan. Finally the experts' opinion was considered. For years they had referred to the necessity of realizing this great project so as to prevent an increased water shortage which would also affect even larger areas and be detrimental to public health as well as cause a decline in the productivity of industry and agriculture.

Before starting the project of construction of dams across the Bode Valley (Bodewerk) the water reservoirs existing in other parts of the GDR were considerably improved. The first two dams across

Rappbode Reservoir under construction at night

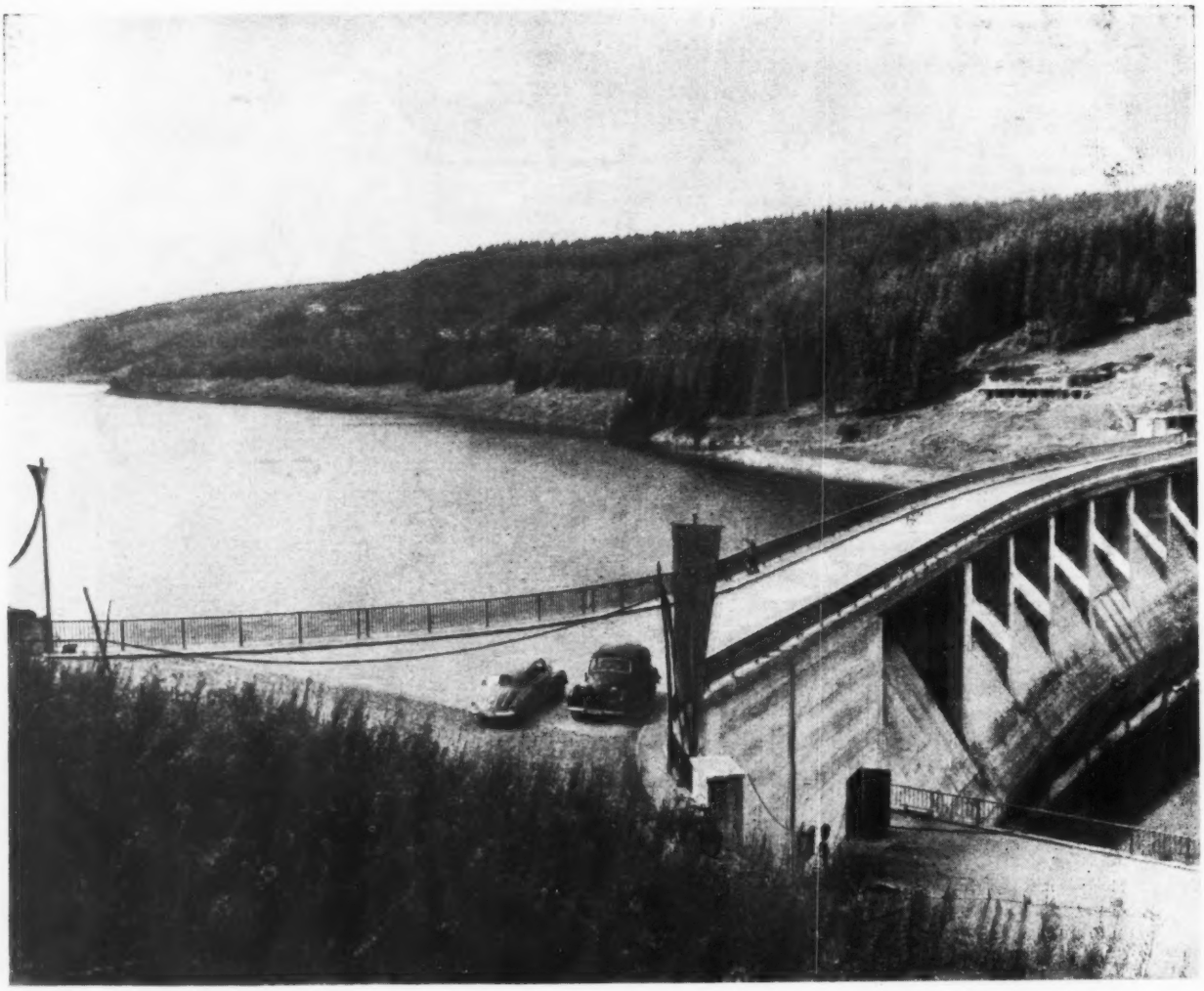
valleys that were constructed in Germany after 1945, were those near Sosa and at Gransahl. After improvement of both supply and consumption of water in the Erzgebirge the first dam across a valley of the eastern Harz was constructed in the Wipper in 1951/52, with a holding capacity of two million cubic meters of water. This plant stores water which is fed into the Wipper during the summer season. The river serves the water supply of the Hettstedt industrial area. The Wipper Dam has already proved to be a bulwark against high water, reducing the high water waves by 45 per cent. Thus great damage was prevented, while at the same time an additional stock of industrial water was provided in the storage basin.

After this first successful beginnings of water regulation, the construction of the huge dam across the Bode Valley was started in 1952, its centre being the Rappbode-Dam. The whole Bodewerk project is a system of several dams across the valley which guarantee a very rational utilisation of water. From the national economic point of view this great project is of essential value. It provides high water protection, a sufficient supply of drinking-water, the irrigation of agricultural areas, the supply of water to our industry and trade and, last but not least, power generation.

The system of dams across the Bode Valley starts at the very spot where the two Bode rivers that stream down from the Harz, flow together and form the Grosse Bode. Here two earth dams are being constructed. Behind them high water protection basins with a holding capacity of upto five million cubic metres of water are being built. They take up the first high waves of the rivers and then transfer the water in as equal quantities as possible to the big dams facing the valley. Near Rübeland the river meets a transfer dam which is 110 meters long and 18 meters high. Here 1.2 million cubic meters of water are being dammed up. Part of this water is transferred to the lower course of the Grosse Bode, while most of the water flows to the main dam through a tunnel of 1.7 kilometers length.

The main dam—the biggest in Germany—is now being completed in the Rappbode Valley between high mountain-ridges. This concrete wall which is 420 meters long and 105 meters high, will dam up a lake of eight kilometers length and a holding capacity of 110 million cubic meters of water. Also on other spots of confluence of this huge dam preliminary dams are being built. They are to purify the water, as the water reservoir is to be used for drinking purposes as well.

Two cubic meters of drinking-water per second can be taken from the storage basin. Through tunnels the water will be transferred to a waterworks. After appropriate processing and purification the water is



Koenigshvette Dam with a length of 130 metres and a holding capacity of 1.2 mil m³ water.

to flow through a long-distance supply line (427 kilometers long) to the districts of Magdeburg and Halle-Merseburg. These industrial centres are to be provided with an annual quantity of 60 million cubic meters of pure drinking-water.

In future the dam will supply five-fold the present quantity of industrial water (water for industry and agriculture which is not suited for human consumption). Upto now the water supply through the Bode river had been very irregular during the dry season. If required, the water supply can be increased by ten fold the quantity of 150 days of the year thereby any further lowering of the ground water level will be prevented.

The Bodewerk will also contribute to power supply. The flow of drinking-water will be transferred from the huge basin via a power station. Power stations will be constructed on the fall-levels behind the individual dams across the valley as well.

A tunnel with a fall of 164 meters is to lead down from the Wendefurth Valley dam to the industrial town of Thale, so as to supply energy for a large power station located there.

The project of the Bodewerk is an exemplary beginning towards the regulation of hydrological conditions, which are of vital importance for the future of a large industrial area of the GDR. That is why the project of the Bode Valley Dam has met with due international interest as a field of research and of technical progress.

Salute to the T.V.A.

The year 1958 completed twentyfive memorable years of the Tennessee Valley Authority. It has not only inspired valley development in the United States ; it has also been an inspiration to similar projects in many parts of the world. Indeed, it has been a great international demonstration project for all these years. We have published elsewhere in this issue an interesting contribution which gives an idea of the impact of the T.V.A. on the world. Many under-developed countries have undertaken water resource development on the model of unified and integral planning which was first initiated by the T.V.A. This approach is now more or less the standard practice. Even a casual perusal of river developments in Russia and China will indicate that even they have not remained impervious to the TVA approach from the technical angle. As a matter of fact, recent developments in these countries indicate that water resource development there is much more thorough, and covers the entire basin from the source to the mouth of the river, be it small, medium or large.

Therefore, it would not be proper to say that T.V.A. is not successful. Rather, on the contrary, its success has inspired the rest of the world.

The significant contribution that TVA has made to river resource development may be briefly mentioned here on this occasion. (1) The techniques of integrated multi-purpose river development, and integrated water control management have had a most widespread recognition. (2) The evolution of the importance of a single watermaster in the distribution of the waters of a river, is the result of TVA experience. (3) The value of a single closely integrated electric power system contributing energy from river basin projects. (4) The value of other aspects of reservoir management such as fish and wild life, water recreation, malaria and water polution abatement, etc. (5) A thorough knowledge of the resources, institutions, economic potentials and the people of the region, as also of an understanding of the inter-relations among these. (6) And understanding of the importance of full regional participation in programmes of development, the techniques and devices in securing and maintaining co-operation, not by "purchase of consent", but by arriving at a common understanding of objectives.

The success of the T.V.A. in promoting unified water resource development opens up an important field of international co-operation. A large river basin that cuts across international boundaries can now be conceived in terms of strictly economic and technical project for mutual benefit of adjacent or successive countries and a non-political river authority can be created. Obviously in the context of the present international suspicion and distrust, such a method will have to encounter many difficulties, but it certainly opens up a positive perspective.

As a result of the success of the TVA in United States several valley authorities have been created in Asia. In India the Damodar Valley Corporation is such an example. But one should not expect a 100% imitation of the TVA in any other country for the simple reason that the historical and social setting out of which the TVA grew can never be found elsewhere. Recognising this fact, however, it is necessary to point out that the essential spirit of the TVA has also not been fully appreciated by the authorities who initially appeared to be enthusiastic about the TVA method. This is one reason why the D.V.C in India created false hopes which crashed into disillusionment subsequently.

One of the major factors for the absence of the TVA spirit in the river valley authorities in India, is the total lack of desire on the part of both the Central and the State Governments to devolve automomous powers to such an authority.

Even in the country of its origin there have been no similar valley authority created. It has been suggested in many quarters that the TVA approach as an organisational method might have succeeded in that region but it does not suit elsewhere. It is, however, regretable that the other side of the medal is not carefully examined. The very success of the TVA and the admirable performance of its allotted tasks has come in the way of this being taken up elsewhere. It has created and aroused new animosities. For example, the privately owed utility industry has been aggressive in attacking the TVA. The State capitals imagine that the valley agency would wield too much power over the economy of the States. A third contributing force to opposition to valley authorities has come from the several agencies with nation-wide responsibilities for resource development or management.

These factors have exercised great influence in the United States. Similar and other influences, which we need not go into here, have also contributed to the creation of valley authorities not based on T.V.A. principles.

The latest trend in the United States appears to weaken the TVA from within. This indeed is extremelyunfortunate, particularly when the TVA has proved as a great beacon light to the rest of the world.

One of the great advantages in the TVAimethod is the reduction of political controversies and influences over river resource development on a nasin-wide basis. As the world becomes more internationally minded and economically inter-dependant, such an approach will become increasingly indispensable We shall be paying our tribute to the TVA if we accept this method and apply it not only in small and medium river basins, but even in the case of large international rivers, such as, the Ganges and the Indus in India.

World Conference on Nuclear Science

The fortnight-long "Atoms for Peace" conference attended by over 5,000 scientists from 66 nations was held at the initiative of the United Nations Organisation at Geneva in September this year. It is the second conference of its kind, the barrier of secrecy having been broken at last in the first conference.

There were 2,200 papers for study by the scientists. The very number indicates the large volume of work that is now occupying scientists all over the world on this new science.

One of the greatest results arising out of this conference is the almost total breakdown of secrecy in atomic research for peaceful uses. The scientists not only spoke freely and without inhibition and disclosed facts at the conference table ; they also continued their informal discussions and exchange of ideas even outside the conference. It was indeed more a cosmopolitan gathering of scientists without barriers and distrust born of national denomination. This spirit of the conference was very well reflected in the President, Prof. Francis Perrin's remarks. He said "Secrecy in this field was harmful, dangerous and foolish. Secrecy forced isolated groups to carry out all the exploratory work which could have been shared by all".

Some of the very interesting points that have emerged out of this distinguished conference are being noted below for general information, although detailed reports are still to come.

1. It was generally assumed that usable thermo-nuclear power was still a long way off, and meanwhile they must not slacken their efforts to produce more efficient fission power plants.

2. There was quite lively controversy between the Soviet and American scientists about the virtue of atomic blasts for making massive excavations. The Soviet scientist contended that such explosions would produce very great radio-activity which would be very dangerous. On the other hand, the American scientist held, that the cost per square yard so excavated, would come to only about 5 cents (25 n.p.) against about $2 (Rs. 10/-) by conventional methods.

3. After receiving detailed reports about the experience from the power reactors already functioning in the USSR, UK, and USA, it was generally found that it functions without great difficulty and more or less satisfactorily. There were also reports about some accidents in such plants both in USSR and U.K. and these only emphasised their need for greater caution.

4. The Swedish scientist reported that a town of a population of 70,000 will be heated by atomic power for the first time in the world.

5. The Soviet Union reported a development of an improved method of smelting steel by using radioactive tracers in smelting furnaces.

6. A session of the conference was devoted to the study of radiation sickness and its possible cure.

7. The very interesting uses of radioactive isotopes were indicated, such as to trace the silting up of a river, destroying pests, increase crops, or even measure the engine wear in cars. An exhibition held at the time of the conference also greatly helped elucidation of other uses.

8. The impact of the atomic age on the present day laws was also a subject for discussion. For example there were some questions relating to the possible consequence of accidents that cannot be answered solely from the experience of safe operation. The uncertainty in prediction of hazards in this context is so large that good decisions regarding public safety is difficult to make. Laws against radiation injuries will have to take into account effects that may take 10, 20, 30 years after the exposure to damaging rays. Statutes of limitation may be affected.

9. On several occasions the discussion centered around cost of atomic power viz. power from conventional sources. At the end of the conference it appeared that the cost of atomic power is sure to equal cost of electricity from conventional fuels very soon—by 1963 in Britain, and may be even cheaper as technology improves. Dr. Bhabha of India took prominent part in the discussion of this question, and he pointed out that many under-developed countries such as India, were already at a stage where they could use atomic energy at a cost comparable with, or even less than of conventional electricity. He argued that it was cheaper for them to import nuclear power stations and fuel because fuel costs were less, although the stations were costlier.

Dr. Bhabha who represented India at this conference, made a strong plea for nuclear power in India. He said India must turn to nuclear power production if the power needs of the 6,000,000 additional population is to be met at the turn of the century. He announced the plan to set up a Uranium mill in Bihar capable of processing 1000 tons of ore a day. He also suggested that plans for one million kW of nuclear power for India by 1966 are in hand.

The president of the conference proposed in his closing speech that a third conference of the kind should be called in 1962.

Letter to the Editor

Dar Sir,

Some time back you published an article on the "Future of the River Valley Projects" by Dr. N, K. Bose. In that articler Dr. Bose made certain statements about Lake Mead and Effects of Hoover Dam.

Recently I had been to USA and visited the Hoover Dam and made certain enquiries in regard to the points mentioned in Dr. Bose's article. A reference was made to the Bureau of Relamation USA, through the TCM, New Delhi. I am placing below an extract of the letter received from the Bureau of Reclamation:

"This is in response to your letter of August 7, 1958, to Mr. L. W. Damours, Chief, Division of Foreign Activities, enquiring about sedimentation in Lake Mead. The article by Dr. N. K. Bose, which you enclosed with your letter was pursued with interest. His statements regarding Lake Mead sedimentation cannot be reconciled with the facts obtained by the Lake Mead Comprehensive Survey of 1948-59.

"Sedimentation surveys of Lake Mead made during 1948 and 1949, by the Geological Survey and the Navy, in co-operation with the Bureau of Reclamation and other agencies, show that 1,426,000 acre-feet of sediments have accumulated during the period 1935 to 1948. This is, of course, the volume of uncompacted sediment as it now lies in the reservoir. As a part of the comprehensive report, no attempt was made to estimate the probable life of the reservoir. But taking into account such contributing factors as compaction of the deposited sediment, increased deposition in the river above the reservoir level, and the decrease in trap efficiency, the probable life of the reservoir would be more than 500 years. None of those studies considered the effect of upstream tributaries and thus extend its useful life by many centuries.

"With the placing in operation of Hoover Dam, the flood control problem on the Colorado River was largely solved. The construction of other storage dams on the river and its tributaries will furnish additional security against recurrence of the disastrous floods of the past. Reduction of the flood hazard, however, does not mean the elimination of all control problems. There remains the necessity for controlling the river by coordinating the operation of the control structures and rectifying and maintaining the river channel between structures.

"Before the construction of Hoover Dam, sediment was transported to the lower reaches of the river in amounts varying from 20 million to 300 million tons per year, part of it be deposited at many points along the river and the remainder being carried to the mouth and deposited in the Gulf. With the construction of Hoover Dam, clear water was discharged into the River below the dam. The scouring power of the clear water is very great, and large quantities of material are picked up and transported farther downstream. However, with no source of sediment from upstream, the river channel below Hoover Dam is rapidly reaching a stage of stabilisation and although some dredging and level work must be a continuing process, the hazards of flood damages of more than minor consequence have been largely eliminated."

I shall be thankful if you kindly publish this letter.

Yours sincerely,
D. Mookerjea
Deputy Chief Engineer,
Barrage and Irrigation, DVC.

Reply by Dr. Bose

I am sorry to note that Shri D. Mukerjea has not been able to get any more information about the silting of the Lake Mead than are generally known. The 1948-49 survey data of Lake Mead carried out under the joint auspices of several U.S.A. Government agencies including the Navy, the Geological Survey, the Coast and Geodetic Survey, and the Bureau of Reclamation, have been analysed by various authorities. I shall quote only from a note by Mr. C. F. Vetter, Chief, Office of River Control, Region 3 Bureau of Reclamation, Boulder City, Nevada, U.S.A, read on February 8, 1952 before the Pacific South West Regional Meeting of the American Geophysical Union, National Research Council. In this note Mr. Vetter first gives an idea about the volume of sediment entering the Lake. "During the period from the time the reservoir first began to fill until the time the survey was done, a total weight of approximately 2,004,000,000 tons of silt entered the reservoir. This weight of silt occupies a space of 1,425,000 acre-feet corresponding roughly to an average unit weight of 64.8 lbs. per cubic foot. The greatest depth of silt deposit was found at the upper reaches of the reservoir, far above the silt pocket (dead storage) provided when the dam was designed, and thus largely within the useful operating volume of the Lake. The maximum depth was found to be 270 ft. Proceeding towards the dam, the depth of silt deposits decreased, but along the original river channel deposits 45 to 160 ft. deep were found almost the entire distance to the dam. It was further determined that at the dam itself the silt deposits had a thickness of 106 ft. Only 42 percent of the total silt volume has been deposited within the designated silt pocket (dead storage) below the lowest outlet gates". Then Mr. Vetter proceeds to give an idea about the deposition of silt at different levels. Compiling these informations we get the following picture about the sedimentation that had taken place in the Lake Mead between 1935 and 1948/49.

Levels	1935 *capacity in million c.ft.*	*Sediment deposition 1935-1948-49 in million c.ft.*
1. 895.0 (dead storage)	2.62	0.6
2. 1162.74 (1st April Reservoir) level	21.547	1.333
3. 1213.74 (1st Aug. Reservoir level)	28.547	1.423
4. 1229.55 (Max. W. level)	31.047	1.426

After giving these figures that tell their own story Mr. Vetter concludes as follows: "One of the principal answers which was desired from the sedimentation survey, was the estimated period which would elapse before the *entire storage volume* of the lake was filled with sediment, thus making it useless for storing flood and for regulating the flow of the river. Taking into account the increasing compaction of the deposited sediment as additional layers are placed on top of earlier deposits, it is estimated that it will be almost 500 years before *all storage capacity* of the lake has been lost. The *useful* capacity of the lake will have been reduced to an embarrasing extent long before this time, however, but it may be assumed with confidence that other storage works wil be constructed upstream from Hoover Dam long before the operating volume is reduced to any important extent".

The above remarks need no explanatory note. The table given will explain. I should only like to note that what we are concerned with is *not* the whole life of a reservoir but *its useful* life. This note of Mr. Vetter's as well as the remarks quoted by Shri. D. Mukerjea point to the extreme importance and urgency of Soil Conservation. Though we may build a number of headwater dams to stop silt coming down to the main dam, their lives will also be limited and in the ultimate analysis the solution to the problem will depend mainly on "Soil Conservation" as I had tried to make out in my paper.

(Dr.) N. K. Bose

24 September, 1958.

Continued from page 4

TVA Around the World

A satirist of contemporary ideologies would certainly find it amusing that these semi-developed countries, some of them addicted to a socialist ideal, have employed a businesslike, corporate organization for their new river developments, while business oriented America leans on the Bureau of Reclamation, the Army Corps of Engineers, and the National Rivers and Harbors Congress. There may even be some wry humor in the profligate manner in which the House and Senate Appropriations Committees still finance domestic dams, as compared to the hard-boiled insistence on results of an international "give-away" agency.

Now Russia, and soon China, will be pushing us hard as exemplars to the nations on how to use the water in their rivers to break loose from the immemorial circle of poverty, wasted resources and human frustration. The dams they are building now are bigger than ours, much bigger. So far, I hope this report makes plain, our example, our people, our ideas have been influential abroad at the level of policy decisions and not just engineering techniques. In that we have a precious advantage over the Russians in any free country. Only satellite countries will do what we, or the Communists, *say* is good for them. Independent countries will accept what we have *demonstrated* is good for us.

Beneath their propaganda, Russia and China appeal to underdeveloped nations as models because they can say, "you can do it, too," What we prosperous Americans are liable to overlook is that the power of TVA to inspire men abroad, now that we we are turning our backs on it at home, comes from precisely the same identification, carried one vital step farther. What the engineers and the lawmakers from poor countries hear when they drive out of Knoxville and stop at a hillside farmhouse is, at bottom this : "We were poor, too, 25 years ago. We always had been. You can do what we did using your river and your soil. But you can do it and keep running your own government, too. In fact, with your demonstration farms and co-ops, and your unions, and your new schools you can do it better that way."

The Next Round of Development

Who will teach the world the lesson for the next round of development : how to organize the programs pioneered by the valley authorities on a national scale, how to reach the little watersheds, how to plan for the giant rivers ; more important, how to select and plan and make use of the whole gamut of resource improvements ?

Electricity plus soviets equals communism, said Lenin. We once showed men from every country to equal democracy. Can we add it sitll ?

Our Contributors

Mr. H. D. Sharma is Executive Engineer, Design Division, Yamuna Project, Uttar Pradesh. He has previously contributed in our journal.

Dr. Ing. P. G. Franke, UNESCO Expert and Guest Professor in the Engineering College, Poona. He has contributed previously in our journal.

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Dr. Henry C. Hart, associate professor of political science at the University of Wisconsin, has long been a specialist on river valley development. He is the author of "The Dark Missouri" and "New India's Rivers." He spent two years, 1952-1954, making a special study of the river systems of India.

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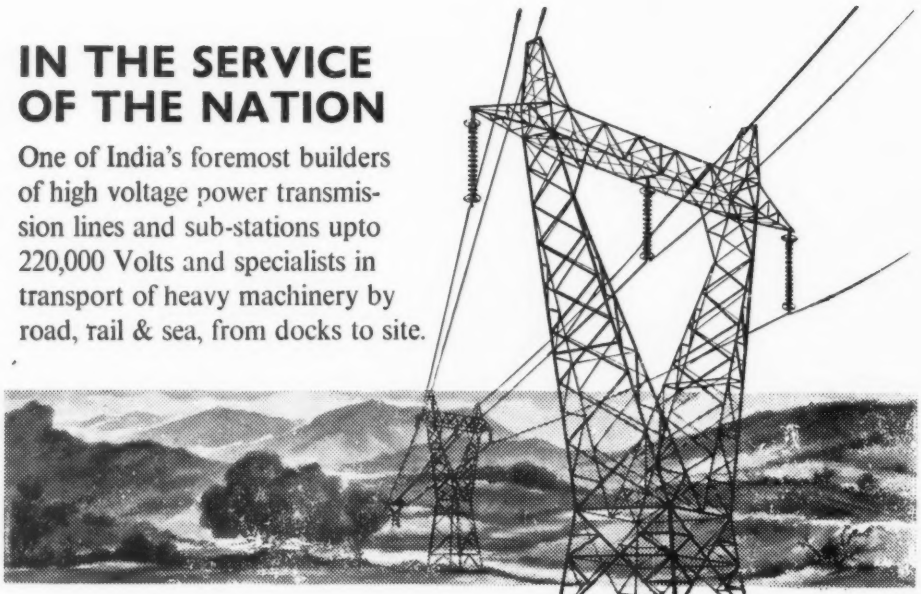

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